

Final Report for:

Arterial Speed Study



Prepared for:
Southern California Association of Governments

Submitted by:

Dowling Associates, Inc.

Transportation Engineering • Planning • Research • Education



In Association With:
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April 18, 2005

Mr. Deng-Bang Lee
Southern California Association of Governments
818 W. 7th Street
Los Angeles, California 93721

Subject: Final Report for Arterial Speed Study

[04-006]

Dear Mr. Lee:

Dowling Associates, Sierra Research and Wiltech are pleased to submit this final report for the Arterial Speed Study.

We would like to thank Deng-Bang Lee, Mike Ainsworth, Hong Kim, Gouxiong Huang, Huasha Liu, Teresa Wang, Dale Iwai, and many others at SCAG for their technical assistance, advice, and review throughout this project. Gouxiong Huang, in particular, advised us on how the link capacities for the model had been determined.

We would like to thank Mr. Verej Janoyan, Ray Andrade and the other staff at the City of Los Angeles Department of Transportation for their assistance in providing real-time traffic and speed data for city streets.

I would like to give credit to Mr. Robert Dulla and Mr. Thomas Carlson of Sierra Research who prepared the sampling plan for the study and the recommended sampling plan for monitoring of regional travel speeds on an on-going basis in the future. Sierra research also provided the GPS equipped laptops, software, and data reduction for acquiring second-by-second data on position and speed for the floating cars.

Dr. Alexander Skabardonis and Dr. Karl Petty provided the necessary software and technology to acquire real-time travel speed, signal timing, and volume speed data from the City of Los Angeles Advanced Traffic Signal Act Control (ATSAC) system.

Mr. Moses Wilson, P.E., of Wiltech led the floating car and traffic count data collection in the field.

Within Dowling Associates, Mr. Chris Ferrell and David Reinke led the data analysis effort.

It has been a pleasure working with you and your staff. We wish you the best as you continue your development of your new regional travel demand model.

Please feel free to contact me with any questions or comments.

Sincerely,

Dowling Associates, Inc.

Richard G. Dowling, Ph.D., P.E.

Principal

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1 INTRODUCTION

1.1 Problem Statement

As the improvement of air quality has become increasingly critical to the SCAG region, the ability of SCAG to accurately predict the impacts of transportation improvements and air quality mitigation measures has become more important. Accurate air quality analysis requires accurate predictions of both traffic volumes and traffic speeds.

Travel demand models, like the one employed by SCAG, have typically focused on predicting demand accurately, not speed. Speed has traditionally been an input that demand modelers adjusted as necessary to improve the accuracy of the volumes forecasted by the model. Consequently federal and state regulatory agencies have shown an increasing interest in the validation of both the volume estimates and the speed estimates produced by demand models.

SCAG already collects a complete set of volume validation data for its travel demand model. What is needed is a cost-effective means for adding speed data to SCAG's regular collection of volume data.

Recent improvements in technology have enabled the automated collection of speed and volume data on freeways. Although not perfect, the PeMS systems enables SCAG and Caltrans to monitor over 263,000 directional-miles of freeways in the SCAG region at very low cost. The driving public can even access the information over the internet on a real time basis to determine congestion problem spots on their drive home from work.

However, the collection of arterial speed data will be quite a bit more difficult than for the freeways for two reasons: 1) SCAG has 9 times more directional miles of arterial streets than freeways, and 2) Arterial speeds fluctuate a great deal more over time and space than freeway speeds.

1.2 Project Objectives

The objectives of this study are to:

- Develop a cost-efficient methodology for gathering speed data for the various levels of arterials throughout the SCAG Region

that is sufficiently accurate for model validation purposes and potential congestion monitoring uses.

- Determine the number of samples necessary to validate the Regional Model's output speeds.
- Conduct a Pilot Survey to demonstrate the practicality of the methodology and begin building the Regional speed database.
- Develop a program to continually gather speed measurements to update the Regional Arterial Speed Database and monitor speed changes over time.

1.3 Project Approach

A pilot study was first conducted to determine the most cost-effective method(s) for collecting speed and volume data for arterial streets in the SCAG region. The pilot study was conducted on Lincoln Boulevard (SR 1) in Los Angeles. Speed and volume data were gathered simultaneously by field observers, floating cars, and in-road loop detectors.

It was determined that in-road loop detectors, when fully operational, can provide speed and volume data almost as accurate as manual counters and floating cars. However, transfer of speed data, flow data, intersection timing data and intersection geometric data from city files for many more arterials would have imposed a significant demand on LADOT staff resources. Consequently, it was decided to conduct the remainder of the speed/volume data collection for SCAG using manual techniques.

Speed and flow data were then gathered for 12.8 directional miles of arterial streets at 8 different locations (including the original Lincoln Blvd. pilot study) in the City of Los Angeles.

This data was used to test and refine various speed-flow curves for the SCAG Model. Additional data was obtained for the I-10 freeway (from a prior study) and the data used to test and refine the SCAG model speed-flow curves for freeways.

A sampling plan for the region was then developed for obtaining data on regional speed trends. A regional speed data collection monitoring program was then prepared for on-going monitoring of regional speed trends.

2 PILOT STUDY

This chapter describes the pilot study and its results.

2.1 Objectives

The primary objective of the pilot study was to test various low cost data collection techniques for measuring speeds against “floating cars”. By collecting speed and other data simultaneously at a pilot test location, different methods of speed estimation can be compared and calibrated to determine the most accurate and cost effective methods available for more widespread use across the SCAG region in the later phases of this project.

The low cost speed measurement techniques tested in the Pilot Study include the following:

1. The APeMS method that relies upon real-time signal loop detector counts and real-time signal timing data typically available from an ATSAC type traffic management center (TMC).
2. Floating cars equipped with GPS-equipped PC's

We had planned to obtain overhead video of the pilot study site, which would have allowed us to measure the accuracy of all four methods against the mean speeds measured in the overhead video. But because of forecasts of bad weather on the day of the pilot study, the overhead video was canceled. Hence, the floating car method is considered to be our “gold standard” against which the accuracy and cost of other methods are to be compared.

2.2 Site Selection

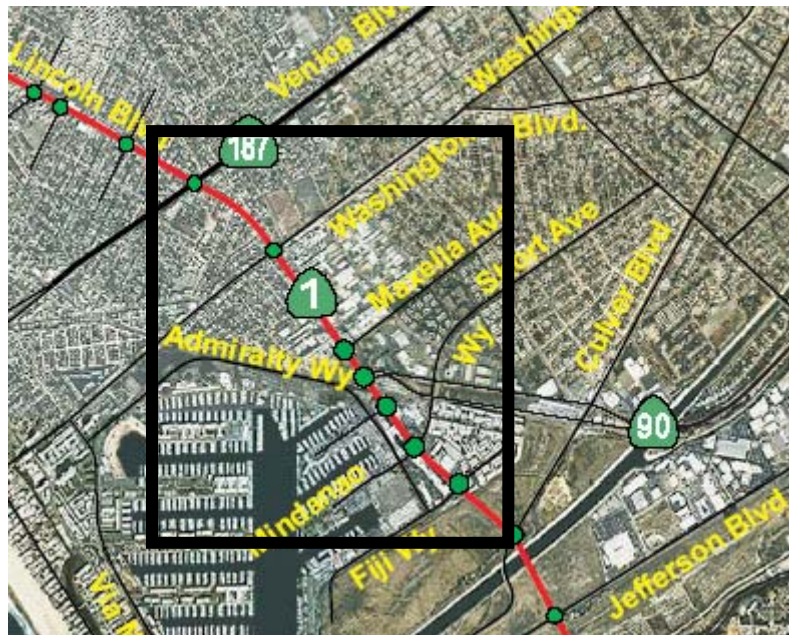
The pilot study was limited to a single site to minimize the cost of testing the other non-floating car methods for the purpose of meeting the “monitoring” objective of the project. This freed up more funds for meeting the SCAG model speed-flow curve calibration objective of the project.

The site for the pilot study was selected based on four criteria:

1. The site arterial should have a full complement of ATSAC detectors that are working and reliable;

2. The traffic and geometry data for the arterial must be readily available (i.e., a readily available, coded CORSIM or Synchro network);
3. The site arterial must have recurrent congestion during the proposed study period; and
4. The study site arterial must be a mile or more in length.

Based upon these criteria, we selected Lincoln Boulevard, between Fiji Way and Venice Boulevard in the Cities of Los Angeles and Santa Monica, for the pilot study (Exhibit 1).



Base map excerpted from Lincoln Corridor Mobility Study, Meyer, Mohaddes Associates, Inc.

Exhibit 1. Lincoln Boulevard Pilot Study Site.

The site is an approximately 1.42-mile long stretch of arterial north of the Los Angeles International Airport. The pilot study segment is a part of a signalized urban arterial with seven signalized intersections with cycle lengths of 120 seconds and links with lengths varying from 500 to 1,600 feet. The number of lanes for through traffic per link is three lanes westbound and eastbound during the peak periods for the length of the study area. East of Fiji (just east of the study area), Lincoln drops to two lanes in the eastbound direction. Additional lanes for turning movements are provided at intersection approaches.

Some special features of this street are:

- A large proportion of turning movements in the study area;

- Some lanes used for through and right turn traffic in the network do not have loop detectors;
- The westbound Lincoln narrows from three to two lanes west of Washington. Since demand is very high during the morning peak hour, vehicles queue upstream from Washington block the outgoing (westbound) flow from the intersection of Maxella; and
- According to the SCAG travel demand model Lincoln's facility type is a Principal Arterial and it resides within an area classified as an Urban Business District.

2.3 Data Collection

The Dowling Team carried out the study on a single day, Wednesday May 26, 2004, between 6 and 10 AM. This long time period enabled us to obtain speed-flow data for low volume pre-peak conditions, peak hour conditions, and post-peak, mid-day flow conditions.

The pilot study collected and cross-referenced data from a number of sources to determine the degree of accuracy and reliability of the various data collection methods employed:

ATSAC data. With loop detectors installed at all intersection approaches in the study area, the ATSAC database can provide volume, occupancy, and loop speed estimate data as well as signal timing data for all the study area (signalized) intersections. The Dowling Team arranged for a team member to visit the ATSAC offices to obtain these data for the study period.

Floating car data. For the pilot study we used three floating cars for the 1.42-mile stretch of road in the study area. On average, a floating car passed the starting point every 7.5 minutes. Floating cars were equipped with on-board GPS receivers that collected location information to cross-reference with the speed data. Wiltec from the Dowling Team provided the probe vehicles and their operators, while Sierra Research provided laptop computers with GPS units for gathering the speed and location data.

Intersection turning movement counts. Turning movement counts were carried out at each study area intersection. These were used to check the accuracy of the ATSAC loop detector counts and were used as inputs for the Highway Capacity Manual (HCM) method for estimating corridor (arterial) levels of service and average speeds. Wiltec provided the staff to perform these counts at all of the intersections in the study area.

Overhead videos. As stated above, we had planned on obtaining overhead aerial videos of the arterial street for a large portion of the study period (three to four hours). The videos would have allowed us

to investigate the differences between the mean speeds and delays reported by the floating cars (which are representative of through traffic only) and the mean speeds and delays for all traffic (turning as well as through traffic). The videos were not crucial to obtaining speed data, but would have enabled us to validate the floating car measurements as well as to cross check the manual turn counts. As stated above, the weather was forecast to be bad for the day of the pilot study. Hence, the overhead flight was canceled.

2.4 Pilot Study Results

The first test was to compare loop counts to manual counts. Exhibit 2 shows a comparison of loop and manual counts by time for 15-minute periods during the AM peak. The loop counts appear to be somewhat higher than manual counts.

An overall comparison of loop vs. manual counts for the pilot study segment is shown in Exhibit 3. The loop counts appear to be approximately 6% higher than the manual counts.

Several analyses were carried out on relationships between observed speeds and V/C ratio. Exhibit 4 shows a plot of floating car link speeds vs. link intersection V/C ratio for Lincoln Blvd. from Fiji Way to Venice Blvd., for both directions. The linear regression line shows a reasonable fit, although there is considerable dispersion among the data points.

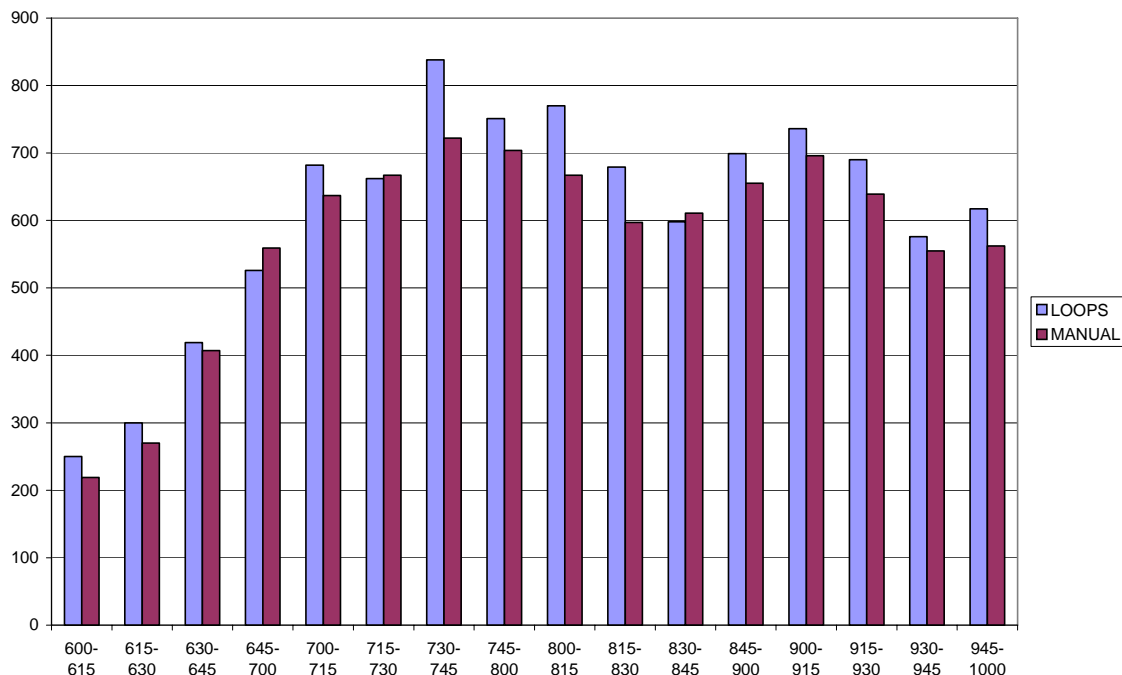


Exhibit 2. Loops vs. manual counts, Lincoln Blvd. at Maxella Ave.

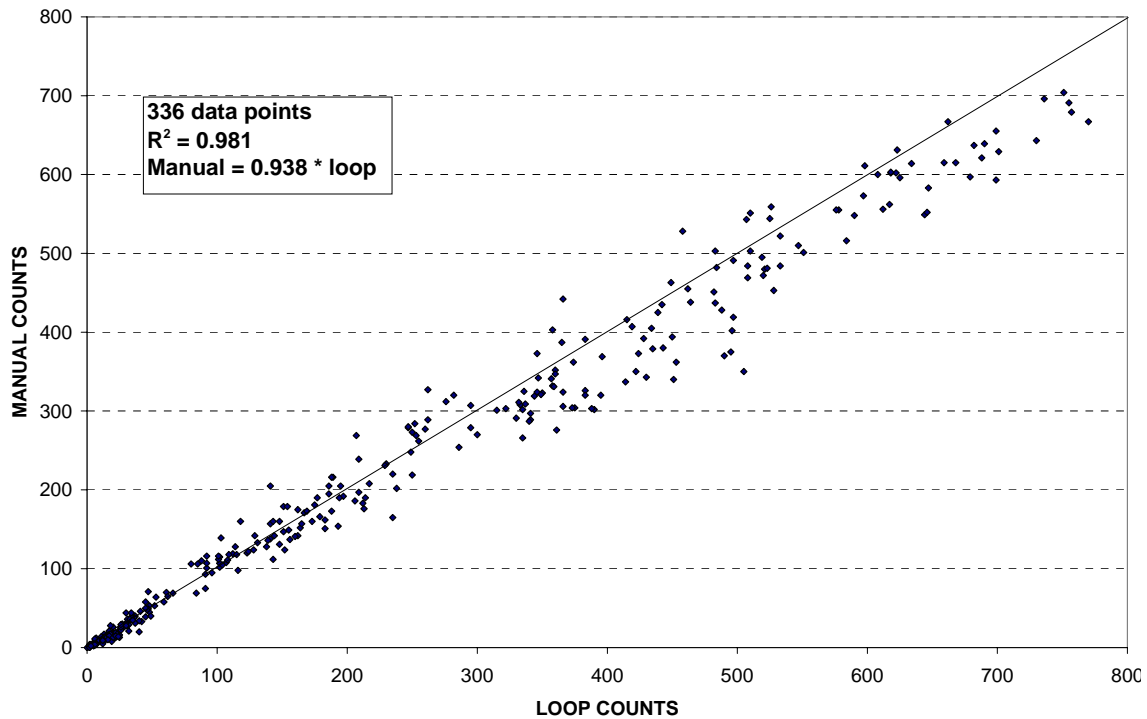


Exhibit 3. Loop vs. manual counts on pilot study segment.

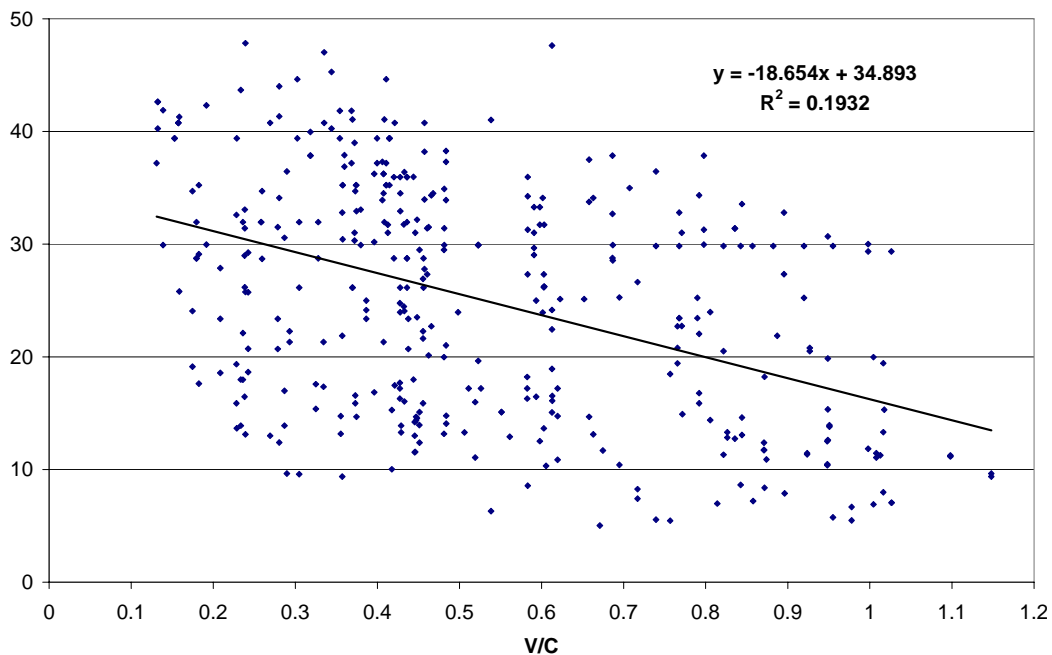


Exhibit 4. Floating car link speeds (mph) as function of link intersection v/c ratio, Lincoln Blvd., Fiji Way to Venice Blvd., NB & SB, 379 data points

Estimating speed vs. V/C relationships by direction did not produce fits as good as those observed for both directions. Exhibit 5 shows the plot for the northbound direction and Exhibit 6 shows the plot for the southbound. Both directional plots show considerably more dispersion than the plot for both directions. Although the regressions produced different values for the slopes of the lines, the slopes are not significantly different from that of the bidirectional line at the 95% level of confidence.

Analyses were also carried out for different sections of the pilot study segment. Exhibit 7 shows a plot of floating car speeds vs. V/C for the section from Fiji Way to Maxella Ave., and Exhibit 8 shows a similar plot for the section from Maxella Ave. to Venice Blvd. Despite the apparently dissimilar slopes of the two regression lines, the slopes do not differ significantly at the 90% level of confidence.

AM travel times along the pilot study segment varied by direction, with the northbound direction showing more congestion than the southbound direction. Exhibit 9 shows the northbound travel times for the AM period and Exhibit 10 shows the southbound travel times for the same period.

Several types of travel time vs. V/C relationships were estimated for each direction. For both the northbound and southbound directions the updated BPR model showed significantly better fits than the alternative models (see Exhibit 11 and Exhibit 12).

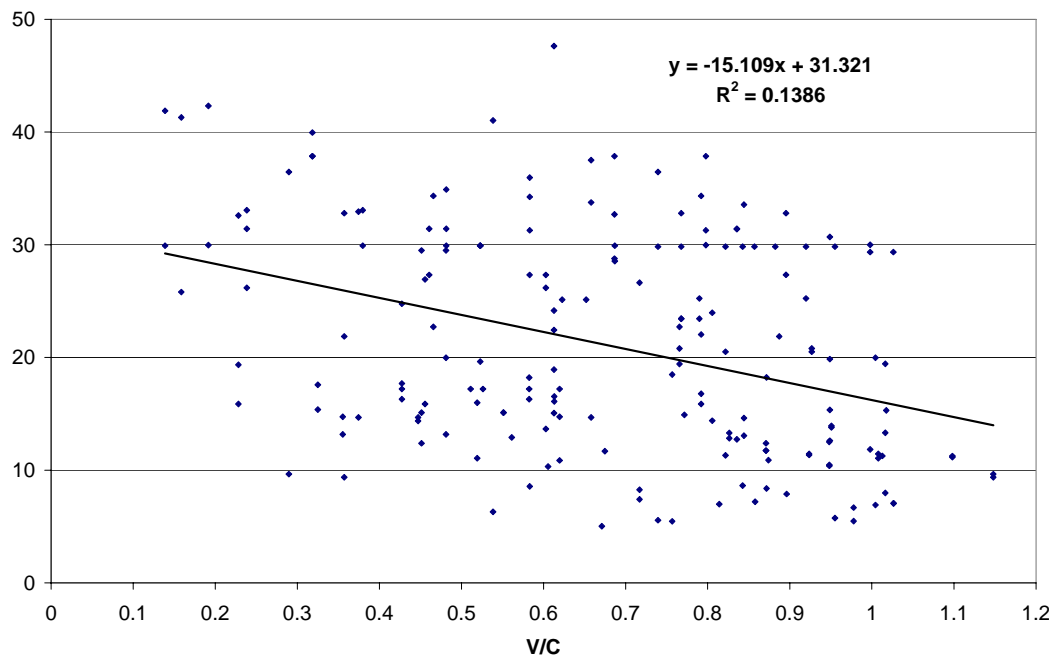


Exhibit 5. Floating car link speeds (mph) as function of link intersection V/C ratio, Lincoln Blvd., Fiji Way to Venice Blvd., northbound.

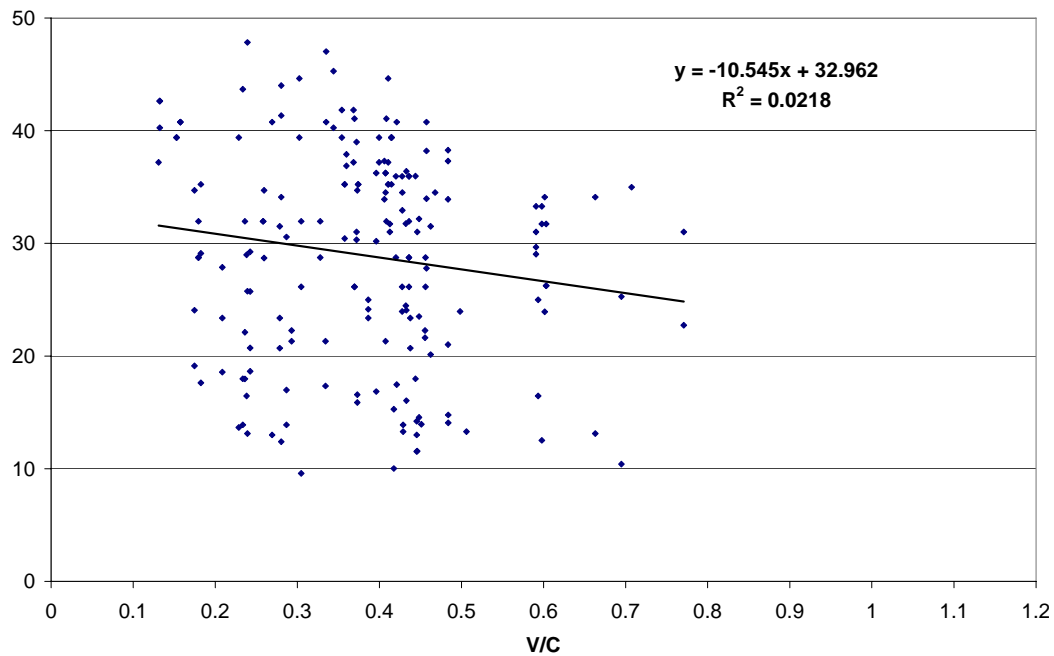


Exhibit 6. Floating car link speeds (mph) as function of link intersection V/C ratio, Lincoln Blvd., Fiji Way to Venice Blvd., southbound.

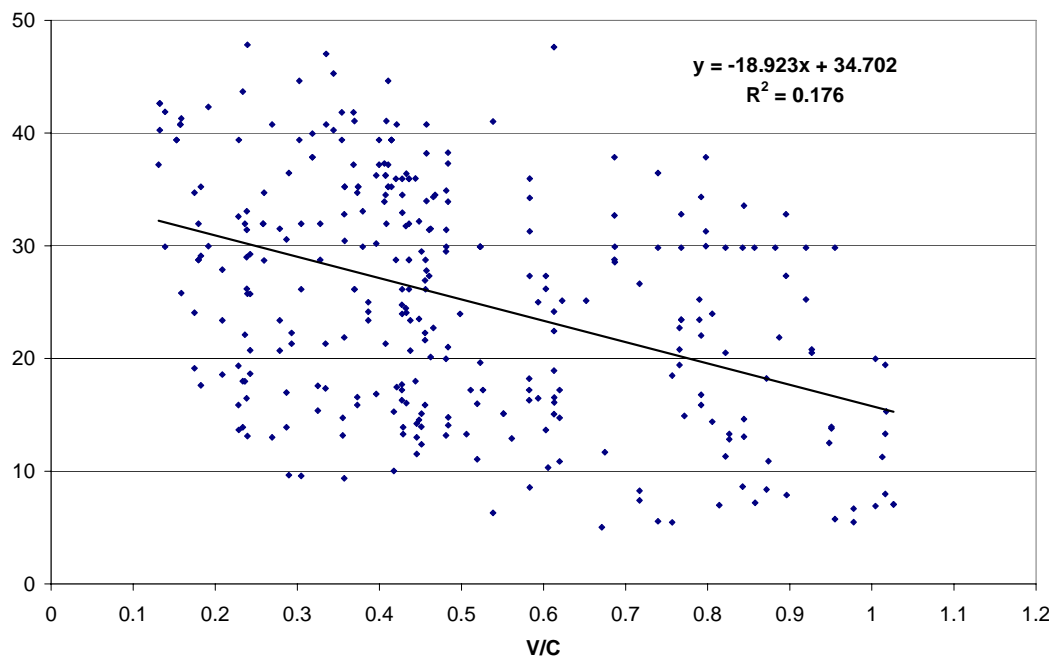


Exhibit 7. Floating car speeds (mph) as function of link intersection V/C ratio, Lincoln Blvd., Fiji Way to Maxella Ave., northbound and southbound.

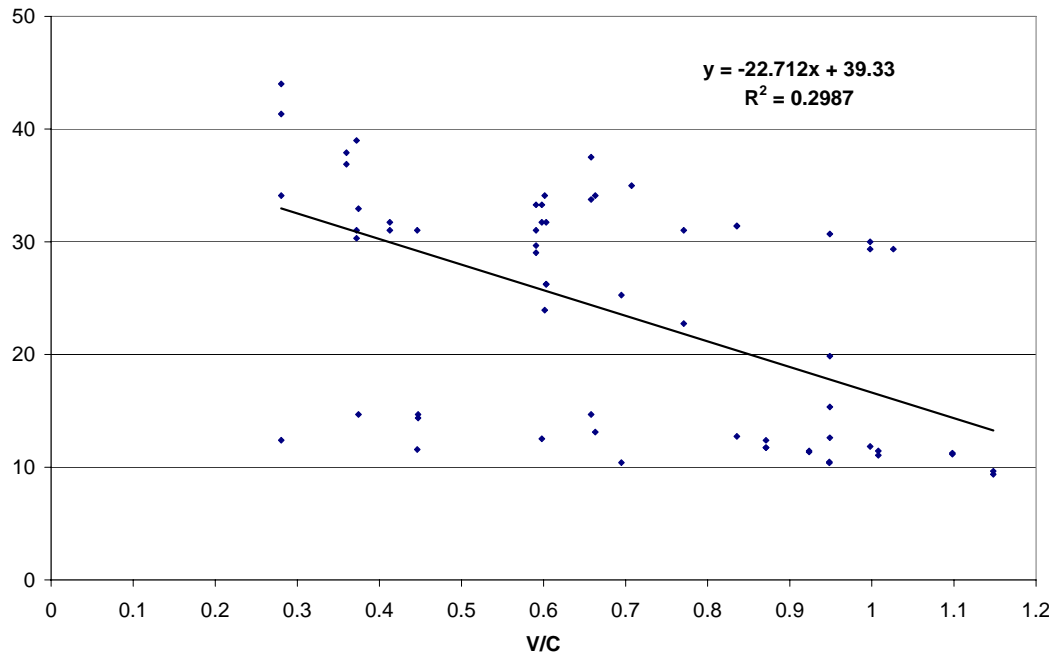


Exhibit 8. Floating car speeds (mph) as function of link intersection V/C ratio, Lincoln Blvd., Maxella Ave. to Venice Blvd., northbound and southbound.

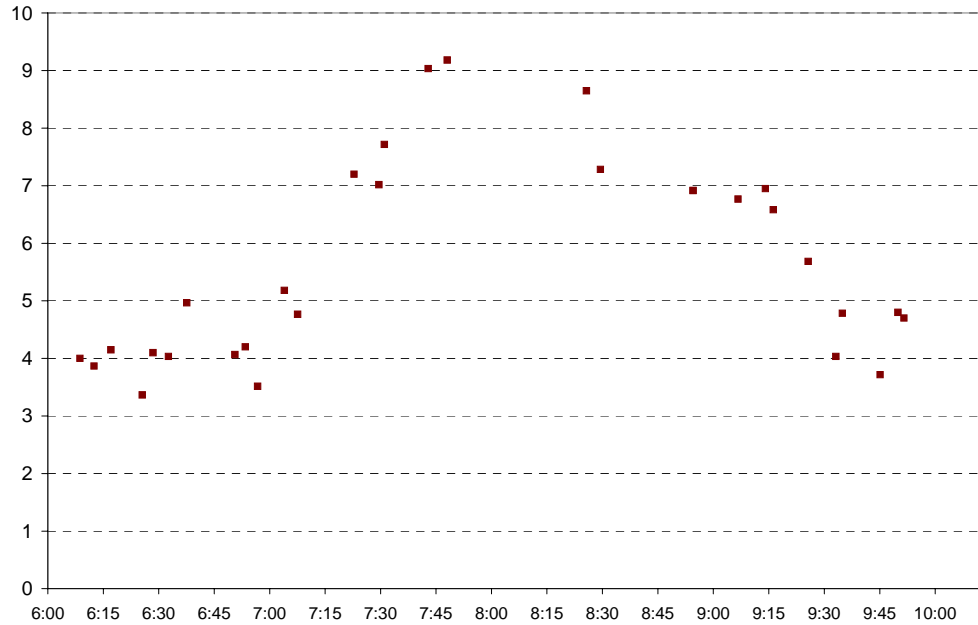


Exhibit 9. Lincoln Blvd., northbound travel time (min) vs. time of day.

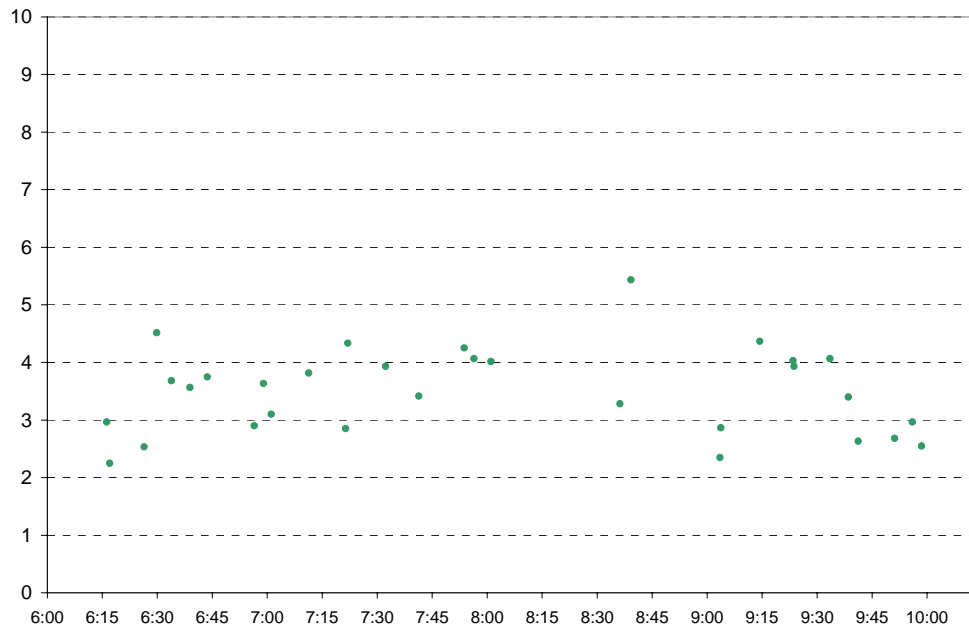


Exhibit 10. Lincoln Blvd., southbound travel time (min) vs. time of day.

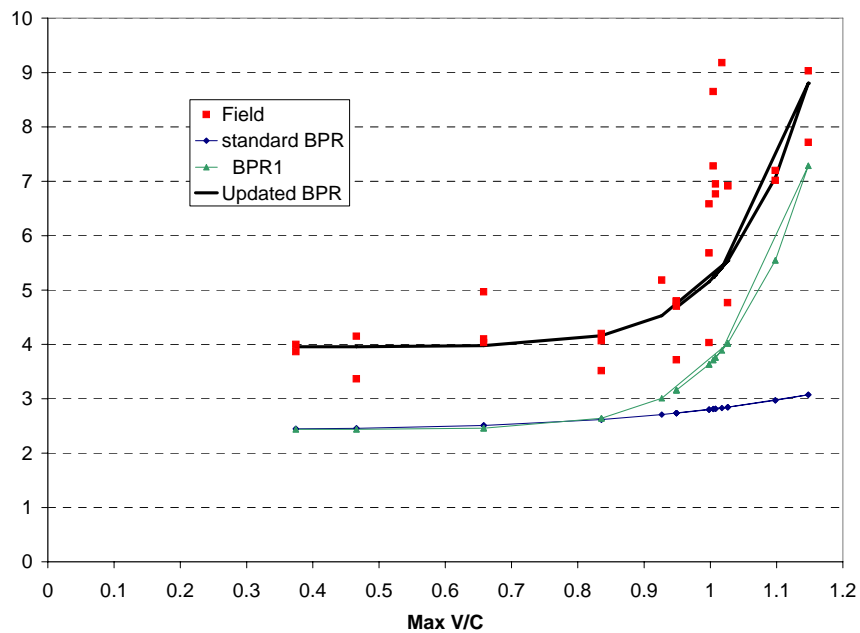


Exhibit 11. Lincoln Blvd., northbound travel time vs. maximum intersection V/C ratio.

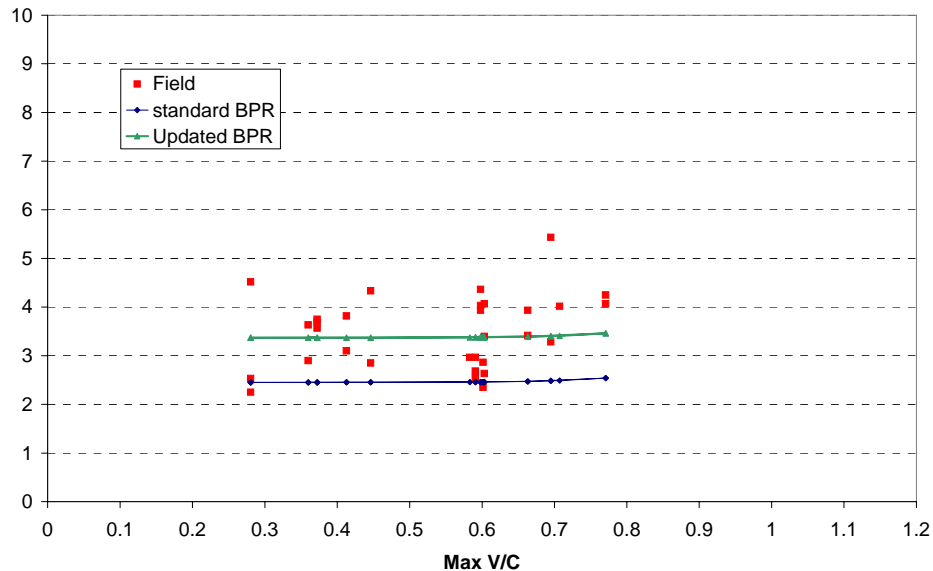


Exhibit 12. Lincoln Blvd., southbound travel time vs. maximum intersection V/C ratio.

2.5 Pilot Study Conclusions

Our comparison of loop detector count data to manual counts leads us to believe that loop detector data will be adequate for use by SCAG for speed monitoring purposes. The approximate 6% over counting by loop detectors in the pilot study can be attributed to physical factors such as vehicles changing lanes at the loop locations, thereby being counted twice.

The overcount factor appears to apply reasonably uniformly across the range of observed counts. Reducing the loop counts by about 6% results in a remarkably good fit to manual counts. Hence, we believe that that applying a fixed “overcount factor” of about 6% to loop counts will provide sufficiently accurate estimates of true traffic volumes.

The speeds measured by the floating cars showed considerable variation even within time periods with similar traffic volumes. None of the speed vs. V/C models that were tested was able to adequately replicate the variations in speed. We therefore conclude that the preferred method for calibrating speed/flow curves for this study is to use floating car speed measurements rather than using speeds derived from speed/flow models and loop detector counts.

3 SPEED-FLOW DATA COLLECTION

3.1 Introduction

The objective of the study was to validate and refine the existing SCAG Model speed-flow curves, capacity look-up tables, and free-flow speed look-up tables. The general approach taken was to first pilot test various data collection technologies, develop a data collection sampling plan, and then to collect speed and flow observations throughout the Los Angeles Basin.

Lincoln Boulevard (SR 1), in the Cities of Los Angeles and Santa Monica was selected for the pilot study. The accuracy of the Automated Traffic Surveillance and Control (ATSAC) System loop detectors was compared to manual turn counts. Floating car measurements of travel time were tested using GPS/PC units. A field measurement approach involving 4-hour long intersection turn counts at all signalized intersections plus GPS instrumented floating cars was adopted for the remainder of the data collection effort based on the results of the pilot study.

Fifteen minute volume counts, and floating car travel times were collected at seven additional sites in the City of Los Angeles. Four continuous hours of data were gathered at each site during either the AM peak (6 AM to 10 AM) or the PM peak (3 PM to 7 PM).

The additional sites were selected within the City of Los Angeles for several reasons. The full variety of facility types and area types within the region could be found within the City of Los Angeles. The City of Los Angeles has real-time monitoring capabilities for most all of its traffic signals thus greatly easing data collection efforts to secure signal timing data. The City and County of Los Angeles account for approximately 58% of the vehicle-miles traveled in the SCAG region.

3.2 Field Data

A total of 216 hourly observations of speed and flow were obtained on 54 directional street segments at 8 different sites in the Los Angeles basin¹. A total of 12.8 directional miles of arterial streets were surveyed. Exhibit 13 below lists the salient characteristics of each survey site.

¹ A street segment is defined as the stretch of street between two signalized intersections.

The surveyed facility types were Principal Arterials (FT2) and Minor Arterials (FT3) ranging from 4 to 6 through lanes (total of both directions), with posted speed limits of 35 and 40 mph. The two-way ADT (Average Daily Traffic) on these facilities varied from 16,000 to 55,000. Individual street segments between signals ranged from a low of one-tenth of one mile up to a high one-half of a mile.

The area types included: Central Business District (AT 2), Urban Business District (AT3), Urban (AT4), and Suburban (AT5).

The v/c ratios (dividing the counted volume by the capacities contained in the SCAG model look-up tables) varied from a low of 0.16 to a high of 2.23. It should be remembered though that it was impossible to count volumes greater than actual capacity in the field because the counting method just counts the number of vehicles crossing the stop bar when the signal turns green (a standard counting procedure) rather than changes in the number of vehicles queued at the signal.

The observed average hourly segment speeds between signals (including the delay at the downstream signal) ranged from 3.7 mph up to 41.1 mph. Individual floating car runs sometimes reached speeds of in excess of 55 mph (even including signal delay) when the car was able to arrive at the downstream signal while it was green.

Exhibit 13. Speed Survey Site Characteristics

Street	From	To	Length (miles)	Facility Type	Area Type	Lanes (both dir.)	Signals (#)	Speed Limit (mph)	ADT (2-way)
1st St	Ford Blvd	Gage Ave	0.90	3	4	4	4	35	19,300
Aviation Blvd	W 120th St	W 135th St	0.99	2	4	4	4	40	33,800
Beverly Blvd	Robertson Blvd	La Cienega Blvd	0.44	2	2	4	4	40	34,700
Lincoln Blvd	Fiji Way	Venice Blvd	1.43	2	3	6	7	35-40	54,600
San Vicente Blvd	Curson Ave	Hauser Blvd	0.64	2	2	6	4	35-40	39,900
Sunset Blvd	N La Brea Ave	N Cherokee Ave	0.51	2	3	6	4	40	33,200
Verdugo Rd	Colorado Blvd	N Shasta Cir	0.83	3	5	4	4	35-40	16,000
Western Ave	W 111th St	W 120th St	0.68	2	4	4-6	4	35	21,300

Facility Type	Area Type
2.= Principal Arterial	2.= Central Business District
3.= Minor Arterial	3.= Urban Business District
	4.= Urban
	5.= Suburban

3.3 Observed Speed-Flow Data

Three plots were created: hourly data, aggregated 3-hour data, and aggregated 4-hour data. The three and four hour aggregations were

created to evaluate the SCAG speed flow curves in their typical 3-hour morning peak period, and 4-hour afternoon peak period applications within the SCAG model

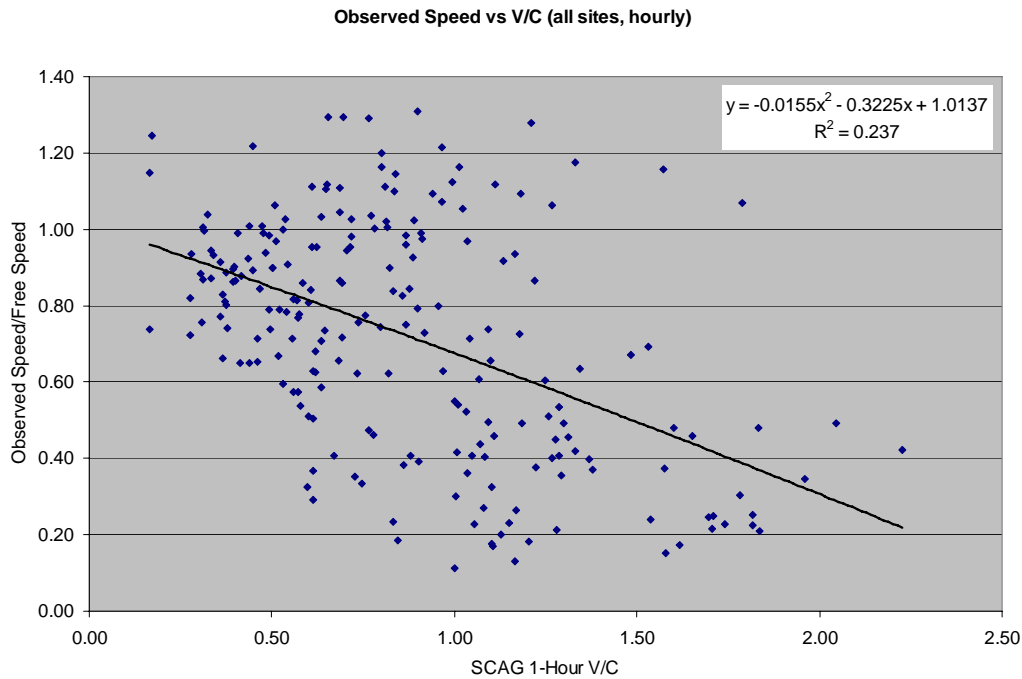
Evaluation of Hourly Data

The 216 hourly speed-flow observations for 54 directional segments are shown in Exhibit 14 below.

All observed speeds were converted to a percentage of SCAG model estimated free-flow speed so that all points could be plotted on the same graph. The SCAG model free-flow speed for each segment was estimated based on the facility type, the area type, and the posted speed limit for each segment, with a 4% increase for divided arterials. The hourly v/c ratios were computed by dividing the counted hourly volume by the SCAG model estimated directional capacity for each segment. The SCAG capacity was estimated based on the facility type, area type, number of through lanes on the segment, and the number of through lanes on the downstream cross street.

While there is quite a bit of scatter, the fitted second order polynomial trend line shows that, on average, the SCAG model estimated free-flow speeds are within 1% of the average of the observed free flow speeds. The mean observed speed at SCAG model capacity ($v/c = 1.00$) is 68% of the SCAG model free-flow speed. The SCAG model v/c ratio explains about 24% (R^2) of the observed variation in hourly speeds.

Exhibit 14. Hourly Speed-Flow Trends

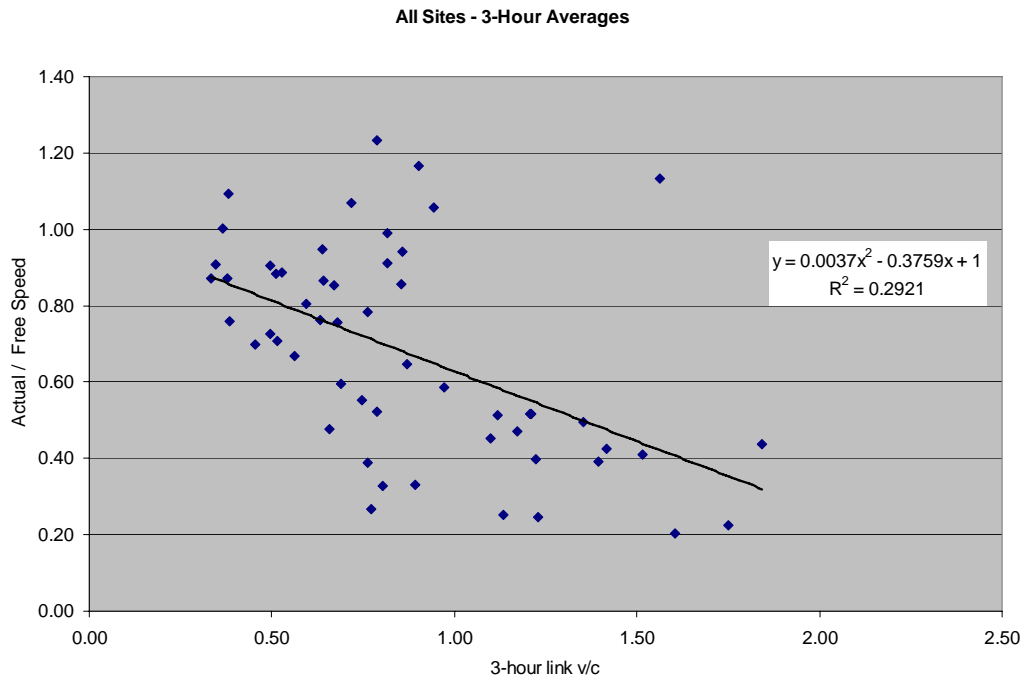


Evaluation of Three-Hour Data

The three-hour data are plotted in Exhibit 15. The first 3 hours of data at each site were aggregated to 3-hour totals. The hourly volume counts were simply summed to obtain three-hour volumes. The hourly average travel times were summed for three hours and then divided by three to obtain the average travel time for the three-hour period. The three-hour average travel time was divided into the segment length to obtain the average segment travel speed for that three-hour period.

The three-hour trendline results are similar to those for the hourly data. The average observed and the average SCAG estimated free-flow speeds are nearly identical. The average observed speed at SCAG estimated capacity is 66% of the SCAG model estimated free-flow speed.

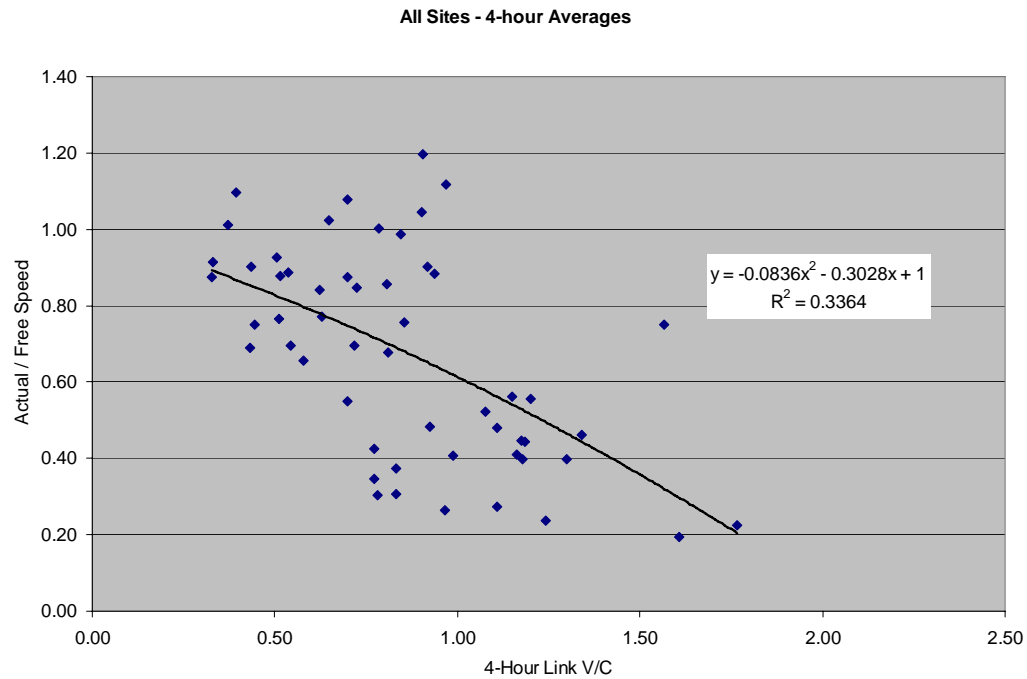
Exhibit 15. Three-Hour Average Speed-Flow Trends



Evaluation of Four-Hour Data

The four-hour data are plotted in Exhibit 16. Again the results are very similar to the hourly data results. One difference is that the range of v/c's narrows. The maximum SCAG estimated v/c for the three or four-hour peak periods does not exceed 2.00. This is to be expected, due to peaking within the peak periods. Higher v/c's may be observed for single hours within the peak period, but the average over the whole period will be lower.

Exhibit 16. Trends in Four-Hour Average Speeds and Flows



4 MODEL SPEED-FLOW EQUATION CALIBRATION

4.1 Introduction

This chapter presents the results of our investigation into the validation and refinement of the existing SCAG Model speed-flow curves for arterial streets.

4.2 Arterial Speed-Flow Curve Options

This section evaluates the existing SCAG model speed-flow equation and other possible equations against the observed field data.

The SCAG Model BPR Curve

The current SCAG model uses the standard BPR curve to estimate link speed as a function of volume/capacity ratio:

$$s = \frac{s_0}{1 + a(cx)^b}$$

where

s = average link speed

s_0 = free-flow link speed

x = volume/capacity ratio

$a = 0.15$

$b = 4.00$

$c = 1/0.75 = 1.3333$

Free-flow speed is determined from a look-up table that SCAG developed to relate posted speed limit and area type to the free-flow speed (see Exhibit 17 below).

Exhibit 17. SCAG Look-Up Table for Free-Flow Speeds (mph)

Posted Speed (mph)	AT1	AT2	AT3	AT4	AT5	AT6	AT7
<i>FT2</i>	<i>Principal Arterial</i>						
20	21	22	22	24	25	27	27
25	23	24	25	27	28	31	31
30	25	26	27	29	31	34	34
35	27	28	29	32	35	38	38
40	28	30	32	34	37	41	41
45	30	32	34	37	40	45	45
50	33	35	37	41	45	51	51
55	34	38	39	44	49	56	56
<i>FT3</i>	<i>Minor Arterial</i>						
20	19	20	21	23	24	27	27
25	21	22	23	25	27	30	30
30	22	24	25	28	30	34	34
35	24	26	27	30	33	37	37
40	25	28	29	32	36	41	41
45	27	29	31	34	38	44	44
50	29	32	33	38	43	50	50
55	30	33	35	40	46	55	55
<i>FT4</i>	<i>Major Collector</i>						
20	17	18	19	21	23	26	26
25	18	20	21	23	26	30	30
30	19	21	22	25	28	33	33
35	20	22	24	27	31	36	36
40	21	24	25	28	33	39	39
45	22	25	26	30	35	43	43
50	23	27	28	33	39	48	48
55	24	28	30	35	42	52	52

Add 5% for divided streets.

Source Table 4.2, SCAG Year 2000 Model Validation and Summary Report

Link capacities are estimated based on the number of mid-block through lanes and the assumed capacity per lane. Capacities per lane are obtained from a SCAG developed look-up table based on facility type, area type, the number of through lanes on the link and the number of through lanes on the downstream cross-street (see Exhibit 18 below).

Exhibit 18. SCAG Look-Up Table for Capacities (vph/lane)

On\Cross	2-lane	4-lane	6-lane	8-lane
<i>AT1</i>	<i>Core</i>			
2-lane	475	425	375	375
4-lane	650	600	500	500
6-lane	825	700	600	550
8-lane	825	700	650	600
<i>AT2</i>	<i>Central Business District</i>			
2-lane	500	450	400	400
4-lane	675	625	500	500
6-lane	850	725	625	575
8-lane	850	725	675	625
<i>AT3</i>	<i>Urban Business District</i>			
2-lane	525	450	400	400
4-lane	700	625	525	525
6-lane	875	750	650	600
8-lane	875	750	700	650
<i>AT4</i>	<i>Urban</i>			
2-lane	550	475	425	425
4-lane	750	675	550	550
6-lane	925	800	675	625
8-lane	925	800	750	675
<i>AT5</i>	<i>Suburban</i>			
2-lane	575	500	425	425
4-lane	750	675	550	550
6-lane	925	800	700	625
8-lane	925	800	750	700
<i>AT6</i>	<i>Rural</i>			
2-lane	575	500	425	425
4-lane	750	675	550	550
6-lane	925	800	700	625
8-lane	925	800	750	700
<i>AT7</i>	<i>Mountain</i>			
2-lane	575	500	425	425
4-lane	750	675	550	550
6-lane	925	800	700	625
8-lane	925	800	750	700

Notes: Capacities are in pcplph.
Lanes are mid-block 2-way lanes.
Add 20% for one-way streets.
Add 5% for divided streets.

Source: Table 4.3, SCAG Year 2000 Model Validation and Summary Report

4.3 Arterial Speed-Flow Curve Options

There are several options for fitting a speed-flow equation to the observed data. The table below describes several options. Speed flow curves however must meet certain requirements in order to permit capacity constrained equilibrium assignment to be performed by the SCAG model. The speed-flow equations must be monotonically decreasing and continuous functions of the volume/capacity ratio (v/c) in order for an equilibrium assignment process to arrive at a single unique solution. As a practical matter, the speed-flow equations should never intersect the x -axis (that is, the predicted speed should never reach precisely zero), so that the computer implementing the SCAG model is never confronted with a “divide by zero” problem.

Exhibit 19. Functional Form Candidates for Speed-Flow Curves

Functional Form	Example	Comments
Linear	$y = -a x + b$	Not acceptable. Reaches zero speed at high v/c .
Logarithmic	$y = -a \ln x + b$	Not acceptable. Has no value at $x = 0$ (the logarithm of “ x ” approaches negative infinity).
Exponential	$y = a s_0 \exp(-bx)$	Has all required traits for equilibrium assignment.
Power	$y = a / x^b$	Not acceptable. It goes to infinity at $v/c = x = 0$.
Polynomial	$y = -ax^2 - bx + c$	Not acceptable. It reaches zero speed at high v/c .
BPR	$y = s_0 / (1 + a (cx)^b)$	Has all required traits for equilibrium assignment.
Akcelik	$y = l / [l / s_0 + 0.25 \{ (cx - 1) + \{ (cx - 1)^2 + 16acx \}^{1/2} \}]$	Has all required traits for equilibrium assignment.

y = predicted speed

x = volume/capacity ratio

a, b, c = global parameters for equation.

l = link length

s_0 = Link free-flow speed.

Cap = link capacity (vph)

Evaluation for Volumes Less Than Capacity

This section evaluates the speed-flow curve candidates against the field measured speeds.

The field data was reviewed and all observed points where demand was greater than capacity were dropped from the data set. This step was necessary to ensure that all observed flow rates and speeds reflected demand on the measured section and not downstream capacity constraints.

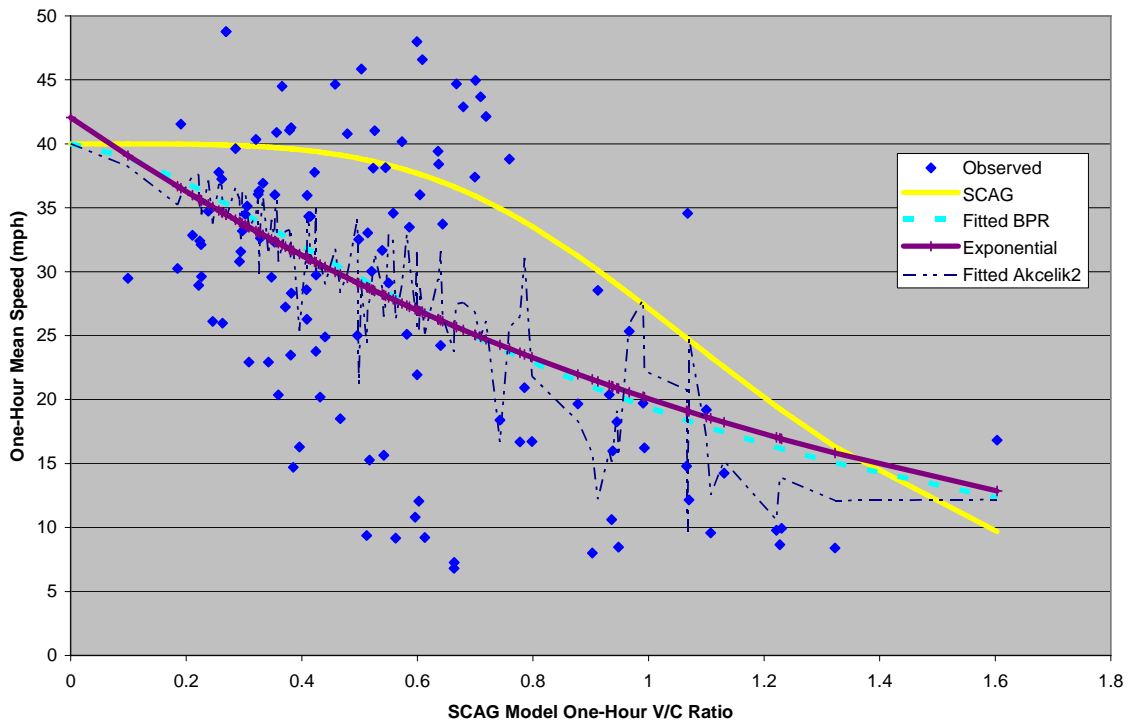
The elimination of congested flow data points was accomplished by eliminating all speed observations for hours when the counted volume was less than the previous hour (such as would occur if traffic backed up from a downstream bottleneck through the intersection). This method also eliminated some observations later in the peak period when the demand may have naturally declined for reasons not related to congestion, but as long as sufficient data points remained for analysis, this was not considered a serious problem. A total 118 observations of hourly volumes and average hourly speeds remained in the data set for evaluation.

All volumes observed in this data set are, by definition, less than capacity, because of the method used to count the vehicles. All vehicles physically able to cross the stop bar of an intersection during each clock hour were counted (This is standard traffic counting procedure). If the vehicles were able to cross the stop bar, then the link approach to the intersection must have had at least that capacity.

Three of the candidate functional forms meet the equilibrium assignment requirements for a speed flow curve: exponential, BPR, and Akcelik. These three equations were fitted through a least squared error fitting process to the observed speed-flow data. Exhibit 20 below compares the fit of the SCAG BPR and the other fitted curves to the data.

As can be seen, the wide scatter of the observed data allows most any speed-flow curve to be drawn through the cloud of data. All three functional forms and the SCAG equation appear to account for some of the observed variation in speeds.

Exhibit 20. Speed-Flow Curve Alternatives Versus One-Hour Field Data



Two Akcelik type speed-flow curves were tested. Akcelik #1 is the original formulation of Akcelik with the v/c ratio (x) divided by capacity within the square root. Akcelik #2 is a simplified formulation without the capacity divisor in the square root.

Akcelik 1:

$$S = \frac{L}{\frac{L}{S_0} + 0.25 \left[cx - 1 + \sqrt{(cx - 1)^2 + \frac{8Jc^2x}{\text{cap}}} \right]}$$

Akcelik 2:

$$S = \frac{L}{\frac{L}{S_0} + 0.25 \left[cx - 1 + \sqrt{(cx - 1)^2 + 16Jc^2x} \right]}$$

Both curves were tested and Akcelik #2 fit the data the best. This curve is plotted in the above exhibit. The Akcelik #1 and #2 curves are of interest because they are not a smooth curve in v/c like the others. The Akcelik curves predicted speed is sensitive to the link length in

addition to the v/c ratio. The Akcelik curves add the same delay to a link for a given v/c ratio, regardless of the link length (The assumption being that all the delay occurs at the downstream signal at the end of the link. There is no delay accruing over the length of the link.) The result is that the Akcelik curve shows a bit more scatter (similar to the observed data) than the other curves for which the predicted speeds are not sensitive to link length.

A statistical comparison of the equations is presented in Exhibit 21. This table shows the root mean square error and the bias for each curve when compared against the observed data. The fitted equations (BPR, Exponential, and Akcelik) naturally do better against the field data than the SCAG equation because they have been fitted to the data. While the SCAG curve over estimates arterial speeds by an average of 7.91 mph (bias), the other curves over estimate arterial speeds by less than one-half mile per hour on average. The root mean square (RMS) error for the SCAG curve is 12.52 mph, while the other curves have slightly lower RMS errors. The best fitting curve, the Akcelik equation, has about a 25% better RMS error than the SCAG equation.

Exhibit 21. Quality of Fit to Observed One-Hour Data

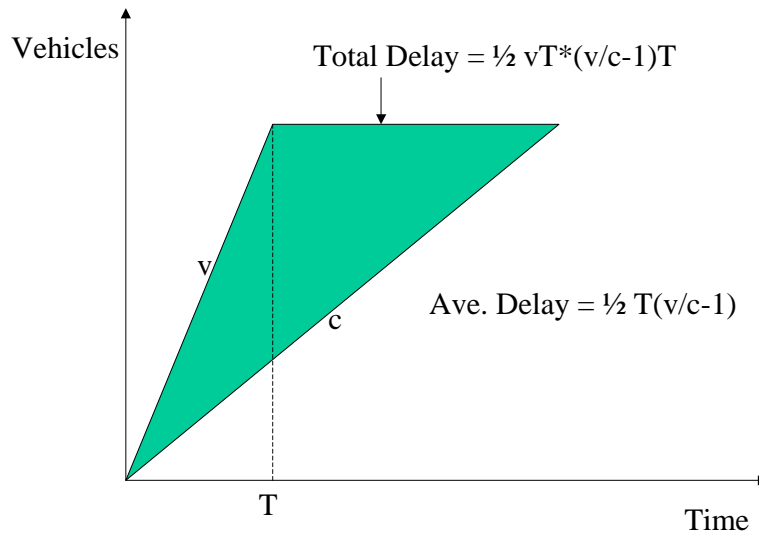
Fitted parameters	SCAG BPR	Fitted BPR	Fitted exponential	Fitted Akcelik 1	Fitted Akcelik 2
a	0.1500	0.7240	1.0512	61.6240	0.0025
b	4.0000	1.5844	-0.7392		
c	1.3333	1.2752		0.3114	0.5273
Quality of fit					
Bias (mph)	7.91	0.30	0.04	0.40	0.10
RMSE (mph)	12.52	9.83	9.84	9.78	9.37

Evaluation for Volumes Greater Than Capacity

The field data could not be used to evaluate the speed-flow curve candidates for demands greater than capacity because the standard traffic counting procedure used could only count the served demand, not the unserved demand. Thus a theoretical evaluation was conducted of the speed-flow curves comparing their predicted delays for volumes greater than capacity against the delays predicted by queuing theory.

According to classical queuing theory, when demand is greater than capacity, vehicles must wait their turn in line until the vehicles in front of them have had a chance to pass through the intersection. This queuing process is illustrated in Exhibit 22 below. The line on the left edge of the triangle is the arrival rate (v), the line on the right edge of the triangle is the discharge capacity of the intersection. The area in between the two lines, the triangle, is the total delay to those vehicles that arrive during time period "T", due to waiting their turn to go through the intersection.

Exhibit 22. Average Delay Per Queuing Theory



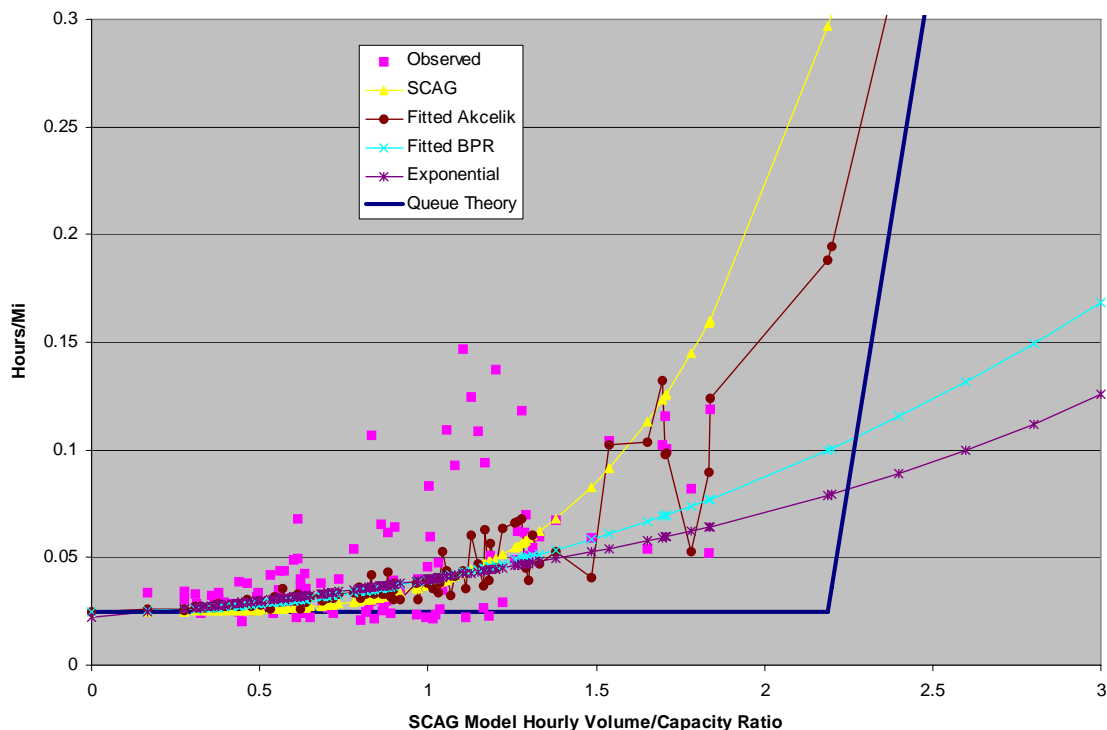
The value “T” is the amount of time that the arrival rate, “v” persists (for example, one hour). The average delay is the total delay divided by the number of vehicles ($v \cdot T$) experiencing the delay. For a one-hour analysis period (T), the average delay is one-half the difference between the v/c ratio and 1.00.

This theoretical average delay can be graphed and compared to the predictions produced by the candidate speed-flow curves. Exhibit 23 below illustrates this (the chart plots travel time per mile, the inverse of speed, so that the theoretical delay due to queuing can be included in the chart). Points that fall on the horizontal portion of the queuing theory line, represent traffic moving at free-flow speeds with no delay. Points above this horizontal line represent speeds below free-flow speeds, with delay.

The theoretical average delay due to queuing is the thick solid line at the bottom of the chart. The line is flat until the real-world capacity of the link is reached², then the predicted travel time increases rapidly, but linearly with increasing demand. The ideal speed-flow curve would not cross this theoretical solid line.

² Because the observed volumes were frequently greater than the SCAG model estimated link capacities, the queuing theory line has been plotted assuming a nominal capacity approximately 2.2 times the SCAG model estimated capacity.

Exhibit 23. Evaluation Against Queuing Theory



As can be seen, two of the candidate curves cross the theoretical queuing delay line, the fitted BPR and the fitted exponential equations. Both of these curves under estimate the delay due to queuing when demand exceeds the real-world capacity of an intersection at the end of a link.

Two of the candidate curves are consistent with the predicted delays due to queuing, the current SCAG BPR curve and the fitted Akcelik curve.

4.4 Freeway Speed-Flow Curve Evaluation

Speed flow data was gathered for 22.0 directional miles of the I-10 freeway in Los Angeles County, between the I-605 and I-710 freeways. This section of I-10 has 4 mixed-flow lanes in each direction and a single HOV lane in each direction.

According to the SCAG model, this section of the I-10 freeway is a facility type 1 (freeway), in area type 4 (urban). The SCAG model capacity for this freeway is 2100 vehicles per hour per lane, or 8400 vph total each direction for all 4 lanes. The SCAG model free-flow speed is 65 mph .

Exhibit 24. SCAG Model Speed/Capacity Look-up Tables for Freeways

	AT1	AT2	AT3	AT4	AT5	AT6	AT7
Speed (mph)	60	62	62	65	65	70	65
Capacity (vphpl)	2100	2100	2100	2100	2100	2100	2100

AT1	Core
AT2	Central Business District
AT3	Urban Business District
AT4	Urban
AT5	Suburban
AT6	Rural
AT7	Mountain

Note: Data for freeway/HOV. Data from Tables 4-1 and 4-5 of SCAG model report.

Volume data were obtained from Caltrans 07 loop detectors for the mixed flow lanes only. The loop detector counts were aggregated to hourly flows. Tachometer equipped floating cars operated by Caltrans District 07 personnel were used to obtain speed data. Approximately three floating car run samples were made for each direction of each freeway segment for each hour. Both volume and speed data were collected for the 4-hour AM peak period extending from 6 AM to 10 AM, on December 3, 2002.

A total of 88 hourly speed and flow observations were obtained over 22 directional freeway segments. The segment lengths ranged from 850 to 8,277 feet. The average segment length was 4,005 feet. Some segments within the 11 mile long study section were dropped from the data set due to count failures at some of the loop detector count stations.

The maximum observed flow rate in a single hour was 2,714 vph per lane. The maximum observed flow rate averaged over the 4-hour period was 2545 vph per lane. Due to normal statistical variation, the largest average flow rate is lower for the 4-hour period than for a one-hour period.

The maximum observed speed averaged over one-hour was 72.4 mph. The maximum observed speed averaged over the four-hour period was 66.7 mph. Due to normal statistical variation, the largest average speed is lower for the 4-hour period than for a one-hour period.

The eastbound datapoints are shown in Exhibit 25. The eastbound direction of the freeway was uncongested in the morning. All but 4 of the data points are above 50 mph.

The westbound datapoints are shown in Exhibit 26. The westbound direction was congested for virtually the entire AM peak period surveyed. Congestion started soon after 6 AM. All but four of the data points fall below 40 mph.

Exhibit 25. I-10 Freeway Eastbound Data

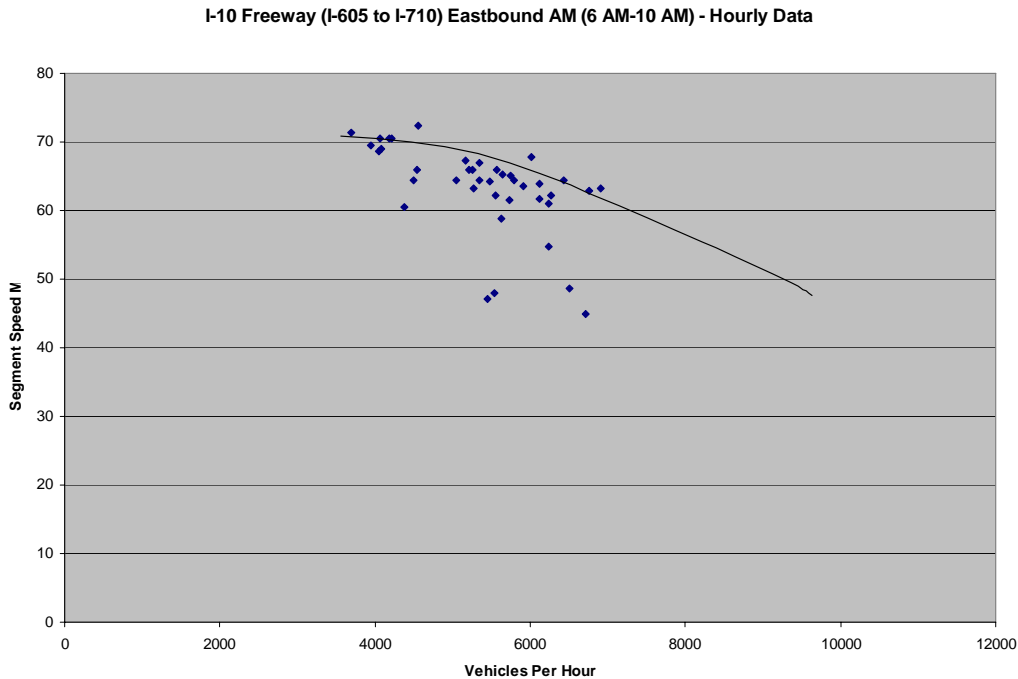
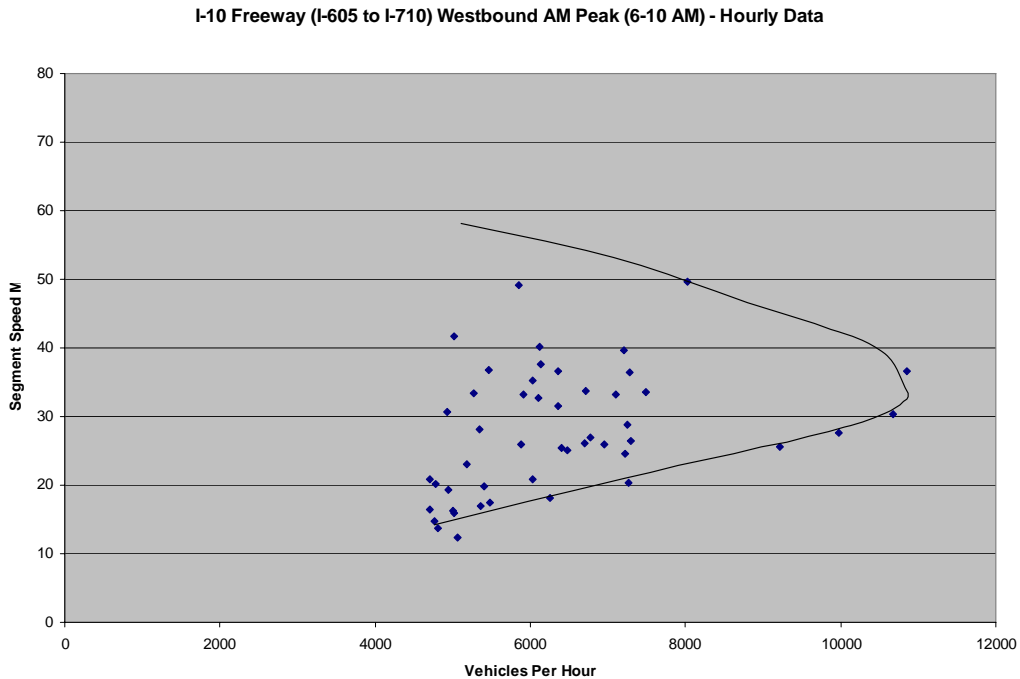


Exhibit 26. I-10 Freeway Westbound Data



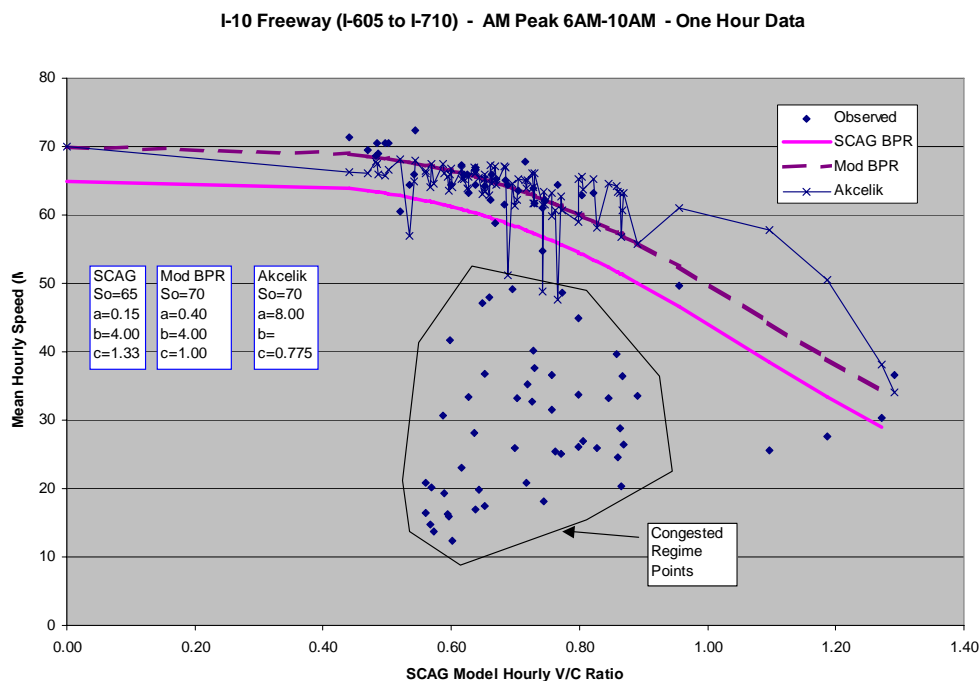
Only uncongested data points could be used for evaluating the speed-flow curve options under demand less than capacity conditions. Thus only eastbound data points could be used.

Evaluation for Volumes Less Than Capacity

Three speed flow curves were tested against the I-10 freeway data set: the original SCAG BPR curve, a modified BPR equation, and the Akcelik curve. Due to the narrow range of volume data available in the I-10 data set for uncongested conditions (only eastbound data could be used), no formal statistical fitting process was applied to the curves. A visual fitting of the modified BPR and Akcelik curves was accomplished, with the objective of matching as nearly as possible the estimated free-flow speed (70 mph) and speed at capacity (35 mph) for the freeway.

Exhibit 27 compares the fit of the SCAG BPR, modified BPR, and Akcelik curves to the observed eastbound and westbound I-10 freeway data. The modified BPR is identical to the SCAG curve, except that it uses a higher 70 mph free-flow speed and the capacity multiplier (1.333=1/0.75) has been subsumed in the “a” parameter value of 0.40. The Akcelik curve has a capacity multiplier of 0.775 = 1/1.29, and an “a” parameter of 8.00.

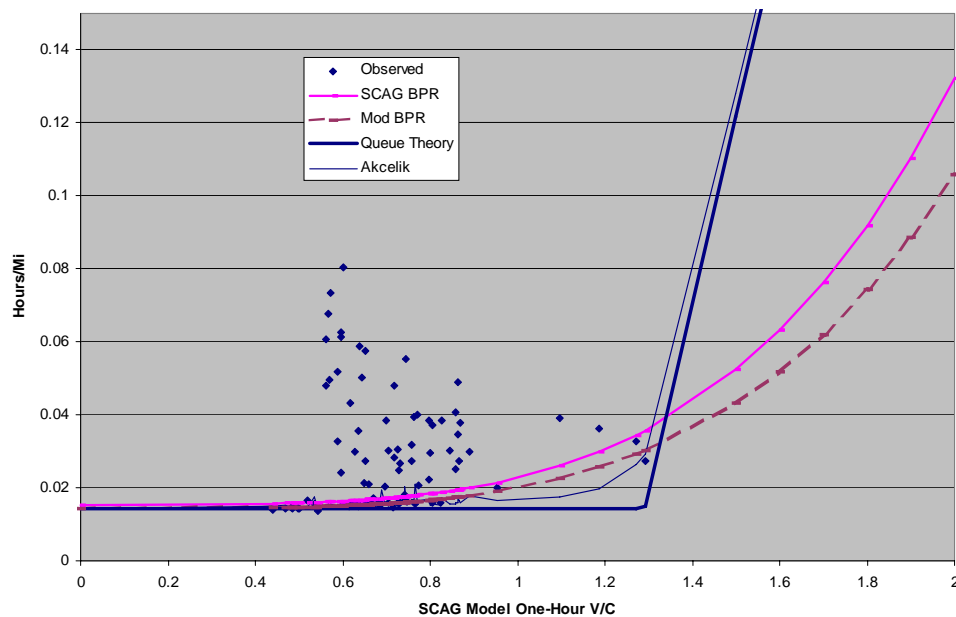
Exhibit 27. Speed-Flow Curves Compared to Freeway Field Data



Evaluation for Volumes Greater Than Capacity

The inverse of the speeds (hours/mile) predicted by the speed-flow curves for demands greater than capacity were plotted against the estimated delays due to classical queuing theory. The results are shown in Exhibit 28. Evaluation Against Queuing Theory. Both the SCAG BPR and the modified BPR curves cross the theoretical delay predicted by classical queuing theory, while the Akcelik equation comes much closer to predicting the delays due to demands greater than capacity. However, at even higher v/c ratios ($v/c > 2.0$) the two BPR curves start increasing faster than the theoretical queue delay curve, until they re-cross that line at around $v/c = 3.5$. Thus the two BPR curves would under-predict queuing delay only for v/c ratios in the range of 1.5 to 3.5. At v/c ratios greater than 3.5, the BPR curves would start over-predicting queue delays.

Exhibit 28. Evaluation Against Queuing Theory



4.5 Practice at Other MPOs

The speed-flow curves used by the following agencies are listed in Exhibit 29:

- Metropolitan Transportation Corporation (MTC), San Francisco-Oakland, California.
- North Central Texas (NCT), Dallas-Fort Worth, Texas.
- Sacramento Area Council of Governments (SACOG), Sacramento California.
- Portland Metro (Metro), Portland, Oregon
- Denver Council of Governments (DRCOG), Denver, Colorado
- San Diego Association of Governments (SANDAG), San Diego, California

Denver COG developed their free-flow speeds from speed surveys during long congestion periods. The free-flow speeds in their model vary by area type and facility type.

San Francisco MTC obtained copies of radar spot speed surveys by the cities of Oakland and Hayward. MTC selected the observed 85 percentile highest speed on the street as the free-flow speed. (This is often also the same speed that a traffic engineer will use to set the speed limit for the street). Since these surveys are spot speed surveys, they measure the highest speeds achieved mid-block and do not include the impacts of any signal delay.

Exhibit 29. Examples of Speed-flow Curves from Other Agencies

Agency	Location	Type of curve	Form	Definitions
MTC	San Francisco, CA	Akcelik	$t = t_0 + 0.25T \left[(x-1) + \sqrt{(x-1)^2 + \frac{8J_a x}{CT}} \right]$	T = flow period (typically one hour) x = degree of saturation (V/C) J_a = delay parameter C = capacity
NCT	Dallas-Ft. Worth, TX	Exponential	$D = \min \left[a \exp \left(b \frac{V}{C} \right), c \right]$	D = delay V = hourly volume C = hourly capacity a, b, c = parameters
SACOG	Sacramento, CA	Conical delay	$T_c = T_0 \times \min \left[\varepsilon - \alpha(1 - \rho x) + \sqrt{\alpha(1-x)^2 + \beta^2}, T_c^{\max} \right]$ $T_c^{\max} = \mu + \nu x$	T_c = congested time T_0 = free-flow time $x = V/C$ $\alpha, \beta, \varepsilon, \mu, \nu, \rho$ = parameters
DRCOG	Denver, CO	Exponential	$C_g = P_{time} T_f (1 + \alpha x^\beta) + P_{dist} D + K$	C_g = generalized cost (\$) P_{time} = value of time T_f = free-flow time x = volume/capacity P_{dist} = value of distance K = fixed penalty α, β = parameters
Metro	Portland, OR	Intersection + mid-block delay	$f_d = \frac{ab + cx^d}{b + cx^d}$	f_d = delay x = volume/capacity a, b, c, d = parameters (vary by midblock or intersection)
SANDAG	San Diego, CA	Intersection + mid-block delay	$f = c_1 T \left(\frac{1}{1 - \frac{c_2}{1 + \exp(c_3 - c_4 x)}} \right)$	f_d = delay T = time x = volume/capacity c_1, c_2, c_3, c_4 = parameters (vary by midblock or intersection)

5 RECOMMENDED SPEED MONITORING PROGRAM

5.1 Introduction

This section outlines a preliminary program plan for on-going collection of arterial speed and volume data. At the outset of this study, SCAG had identified two primary uses for on-going speed measurements data: (1) continued validation of model speeds (e.g., every time a new base year is developed); and (2) historical tracking of speed changes and trends over time throughout the SCAG region.

Prior to the development of the on-going sampling plan, feedback was solicited from key SCAG staff, including the Project Manager and members of the Modeling Task Force, to ensure the plan was designed to meet intended uses and clarify agency responsibilities. A key element of this discussion with SCAG staff related to the second intended use listed above, historical speed tracking. One of the primary goals of the Arterial Speed study was to determine if other measurement technologies existed as a viable, valid and cheaper alternative to “traditional” measurement of arterial link speeds using floating car techniques. At the outset of the study, it was believed that such a technology, if it existed, would be implemented within the on-going sampling program in model speed validation and more broadly, in historical tracking of speeds over the entire arterial network.

Unfortunately, other potential technologies examined under this study (such as aerial photography, arterial PeMS, etc.) were found to be either more expensive or unreliable in accurately measuring arterial link speeds. As a result, SCAG agreed that since no alternative technology currently exists to provide inexpensive and accurate speed measurements over a broad sample of arterial links in the region, the on-going sampling plan should initially provide for continued validation of model speeds using floating car-based measurements. Thus, the on-going sampling plan described in this memorandum focuses on model speed validation requirements. Although it does not rigorously outline costs and program elements for region-wide historical speed tracking, it includes a discussion of various approaches to broader arterial network sampling that can be implemented using existing floating car-based measurements as a “starting point” for a historical arterial speed database.

The remainder of this section presents a framework for sample design in future studies that will support collection of arterial speed measurement and development of updated model speed-flow curves

for use in new “base year” model validation runs prepared each three-year Regional Transportation Plan (RTP) cycle. The following section briefly reviews the floating car speed data collected in 2004, including the precision with which speeds on individual links can be estimated and the variance components encountered in such data. The “Speed-Flow Curves Sampling Plan” section presents a sampling plan for repeating the 2004 study in future years, including estimates of the tradeoff among sample size, precision, and cost. The section entitled “Proportionate Arterial Speed Sampling” considers an alternative sampling plan in which floating car data would be collected in a proportionate manner to permit estimating the average speed on the SCAG system. The final section presents alternatives and recommendations for consideration by SCAG.

Based on feedback from SCAG, cost estimates provided in this report assume that all elements of the on-going sampling plan would be contracted out by SCAG. SCAG staff would only provide oversight and review roles for each work element. Unit costs for various elements of arterial speed and traffic count data collection, post-processing and analysis were estimated from other tasks in this study. Our cost estimate of \$35,000 per study for speed-flow curve development is a “placeholder” estimate based on the Task 8 budget contained in the Project Work Program for the study. SCAG staff can modify these estimates as needed to reflect local conditions and costs.

5.2 Review of Collected Data

During 2004, floating car speed data and co-sampled traffic counts were collected on 10 different arterial corridors in Los Angeles County to permit development of speed-flow curves for the SCAG transportation planning model. The data were initially collected on one corridor in the Pilot Study; 9 corridors were later added to the sample. The data collection was carried out as follows:

- Arterials were identified that could be driven in a complete circuit, with the floating car moving “with” congested traffic in one direction and “against” it in the other direction.
- The arterials consisted of 6 signalized links on the one corridor covered in the Pilot Study and of 3 signalized links in the remainder of the study. In addition to the floating car speed data, vehicle count data were taken for each link.
- The arterials were chosen so that they crossed an area type boundary at one point along the route, thereby sampling conditions in two different area types.
- The arterials and locations were chosen so that the overall distribution of floating car mileage by facility and area type would approximate the distribution of VMT by facility and area

type on the SCAG system. However, only corridors in Los Angeles County were considered.

- Arterials were selected for sampling using pre-existing screenline data in a way to give the most even coverage of the range of V/C (flow/capacity) values. This targeted selection gives the data increased power for determining the shape of the speed-flow curve, particularly in regard to the location of the “knee” where increasing vehicle volume has the greatest effect on speeds. A proportionate sampling strategy would tend to give a bi-modal result showing high congestion in the direction “with” traffic and low congestion in the direction “against” traffic.
- Floating cars were driven on roundtrip circuits for 4 consecutive hours at each selected corridor. Both AM and PM peak periods were sampled in equal numbers.

The rationale for this approach and the methods for selecting suitable arterials were described in a technical memorandum.

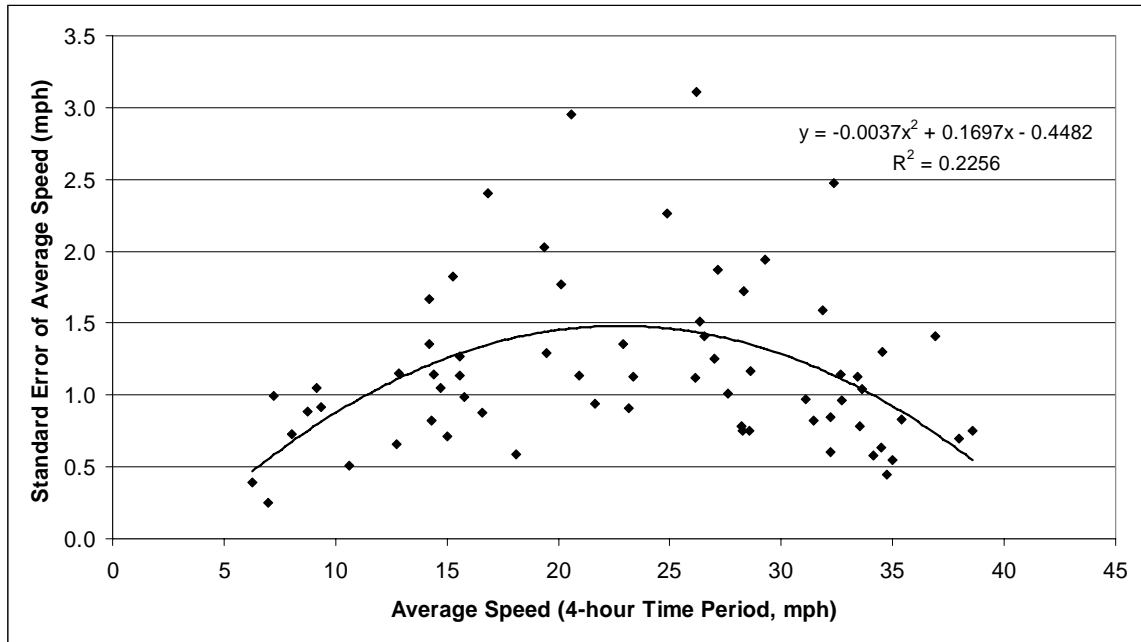
The data collected in the recent study were examined to determine the variation in actual roadway speeds as the most important input to sample size calculations in this paper. For each of the 66 bi-directional links driven in the study, an average speed has been computed over the four-hour period of data collection. The standard error of the average speed was also computed to give the precision with which floating car data can characterize average speeds on a link. Where heavy traffic congestion leads to low speeds, the expected standard error (the curvilinear trend in the figure) is 1.0 mph or less. Although the relative precision is poor (the error is a large percentage of the average speed), the absolute precision is still good. Where low congestion permits relatively high speeds, the standard error is also 1.0 mph or less. In this case, traffic may travel at more constant speeds and the floating cars can complete more roundtrips each hour. The largest standard errors (lowest precision) occur at intermediate speeds between approximately 15 mph and 30 mph. Here, it is likely that congestion levels vary significantly during the four-hour period, leading to an increased variability in travel speeds. Overall, floating car data can characterize the four-hour average speed on a link to within a precision of approximately 1.2 mph.

Exhibit 30 shows the curvilinear relationship between the standard error of estimate and the four-hour average speed on links.

Where heavy traffic congestion leads to low speeds, the expected standard error (the curvilinear trend in the figure) is 1.0 mph or less. Although the relative precision is poor (the error is a large percentage of the average speed), the absolute precision is still good. Where low congestion permits relatively high speeds, the standard error is also 1.0 mph or less. In this case, traffic may travel at more constant

speeds and the floating cars can complete more roundtrips each hour. The largest standard errors (lowest precision) occur at intermediate speeds between approximately 15 mph and 30 mph. Here, it is likely that congestion levels vary significantly during the four-hour period, leading to an increased variability in travel speeds. Overall, floating car data can characterize the four-hour average speed on a link to within a precision of approximately 1.2 mph.

Exhibit 30. Relationship of Standard Error of Estimate to Measured Speed



The variation encountered in the speed data is the result of a hierarchy of average speeds surrounding the system-wide average speed:

- An average speed for each cell formed by the combinations of facility type and area type. These average speeds vary around the system-wide average speed.
- An average speed for each distinct bi-directional link within a facility and area type cell.
- An average speed for each traverse of an individual bi-directional link.

The first two levels in the hierarchy are termed fixed effects, because the average speed is fully determined (if not yet measured) once a particular choice is made for the facility/area type cell and link. The third level is termed a random effect, because it represents the random variation of individual traverses of a link. The overall problem is a mixed effects model in which both fixed and random effects occur.

A components-of-variance analysis was conducted of the four-hour average speed data to estimate the values for the individual variance components in the hierarchy of speeds. The results of the analysis are reported in Exhibit 31.

Exhibit 31. Components of Variance for Floating Car Speed Data

	Degrees of Freedom	Variance (σ^2)	Standard Deviation (σ , mph)
1. Among Facility/Area Types (σ_1^2)	7	3.6	1.9
2. Among Links Within Facility/Area Type (σ_2^2)	54	68.0	8.2
3. Among Traverses within Links (σ_3^2)	2730	52.3	7.2

The results indicate that the average speed varies among links within a facility/area type cell by somewhat more than the speeds of repeat traverses vary within a particular link, while there is much less variation in average speed among the facility and area type cells. The high variation among links within facility/area type is caused by a variety of factors, including the definition of links on a bi-directional basis so that the directions “with” and “against” traffic are counted as separate links, by the different time periods (AM or PM peak) at which links were sampled, and by factors specific to the individual links.

The power of a components-of-variance analysis is that its results can be used in the calculation of the variance that would be expected in a new sample of data. In this instance, if n_1 , n_2 , and n_3 are, respectively, the number of distinct facility/area type cells, bidirectional links, and traverses contained in a sample, then the variance of the sample mean can be computed as given in Equation (1).

$$\sigma^2 = \frac{\sigma_1^2}{n_1} + \frac{\sigma_2^2}{n_2} + \frac{\sigma_3^2}{n_3} \quad (1)$$

The following sections will use these components in estimating sample sizes for future studies to update the speed-flow data. However, these components represent the total variation in roadway speeds, of which part can be accounted for by the systematic relationship between speed and traffic flow. Therefore, it will prove necessary to make adjustments to the estimated values in this sample to better reflect the variation of the quantity being estimated.

5.3 Basis for Cost Estimation

In the following sections we present discussions of two sampling approaches. The discussions include cost estimates for different sample sizes.

Assuming that two floating cars would be driven for 4 consecutive hours at each arterial corridor selected for study, the unit costs used to estimate total speed-flow data collection costs are presented in Exhibit 32.

Exhibit 32. Unit Cost Assumptions for Data Collection and Analysis

Item	Cost
1. Field data collection	\$3,000 per corridor
2. Data post-processing	\$1,000 per corridor
3. Speed-flow curve development	\$35,000 per study
4. Statistical design/analysis	\$5,000 to \$10,000 per study

Note: Field data collection includes hourly traffic counts plus floating car observations.

5.4 Speed-Flow Curves Sampling (Approach I)

The minimum study required for updating speed-flow curves would entail a repeat of the data collection performed during 2004. The methods of data collection would be the same, and as long as the number of arterials sampled is comparable to that of the recently completed study (or larger), a future study would obtain a similar level of precision (or better) in estimating new speed-flow curves for the SCAG model.

Base Study Design

In a base study to update the speed-flow curves, the arterials could be selected without regard to the roadways that were sampled during 2004. As will be explained in later sections, the sample resources can be allocated in more efficient ways, while still satisfying the minimum objective of updating the speed-flow curves. Nevertheless, a base study provides a point of departure in understanding the tradeoff between sample size and precision.

Using the components of variance derived from the data collected in 2004 (and described in the preceding section), we can make an estimate of the precision with which speed-flow curves can be estimated as a function of sample size. The estimate depends on assumptions about the extent to which the observed variance in speed

will be accounted for systematically by the speed-flow curves (as the effect of varying V/C conditions) versus the extent to which the variance will contribute to residual variation around the curves. The estimate will provide useful guidance on the tradeoff between precision and sample size.

The components-of-variance analysis partitioned the total variance in the floating car speed data into three variance components:

- Component 1 (σ_1^2) - variation of average speeds among the cells formed by the combination of facility and area type;
- Component 2 (σ_2^2) - variation of average speeds among bi-directional links that belong to the same facility/area type cell; and
- Component 3 (σ_3^2) - variation of speeds among the individual floating car traverses of each bi-directional link.

The variation of Component 2 is caused both by differences in V/C conditions on the various links (which would be accounted for by speed-flow curves) and by the extent to which otherwise similar links would differ from each other in ways not accounted for by the speed-flow curves. The variation of component 1 is related to a large number of factors that will cause the speed-flow relationship to vary by facility and area type; the diversity of the causal factors (if they are significant) would typically be accounted for in modeling exercises by specifying separate speed-flow curves by facility and area type.

To estimate the precision with which speed flow curves can be determined, it is necessary to compute a value for the residual (unexplained) variation in speeds around the speed-flow curve. The residual variation will measure the vertical uncertainty in the position of the speed-flow curve on a graph. For this purpose we will assume that the functional form of the speed-flow curve does not introduce a lack-of-fit bias to the representation of the speed-flow relationship. We further assume:

- All of the Component 1 variation (facility and area types) will be accounted for systematically by specifying different speed-flow curves for each cell. None of the variance will contribute to the residual variation in speeds.
- One-half (50%) of the Component 2 variation (link differences within facility/area type cells) will be accounted for as the result of varying V/C conditions, and one-half will be associated with unexplained differences among otherwise similar links that will contribute to the residual variation in speeds.
- None of the Component 3 variation will be accounted for by varying V/C conditions, so that all of the variation will contribute to the residual variation in speeds. This effectively

assumes that V/C conditions will remain constant across the four-hour sampling period. In reality, V/C does vary across the multi-hour modeling periods (e.g., AM peak, PM peak etc.). However, this actual variation will be systematically accounted for later by the development of speed-flow curves that represent averages over the multi-hour sampling period. Thus, we can safely ignore the effect of varying V/C on Component 3 variation in estimating sampling requirements under this approach.

The residual variation resulting from one-half of Component 2 can be attenuated by sampling at more locations, thereby increasing the number of individual arterial corridors that are covered. The residual variation resulting from Component 3 is attenuated by increasing the number of traverses made by floating cars on each bidirectional link.

Sierra Research estimates that the most cost-effective approach is to sample arterial corridors that can be driven in a roundtrip transit time of approximately 15 minutes, including turnaround time. This corresponds to corridors with 5 links assuming an average speed of 25 mph and a distance of 0.5 mile per link. If two floating cars are used, then 32 traverses can be made of each bi-directional link in each four-hour period. Given these assumptions, Sierra estimates an average cost of \$4,000 per corridor for in-field data collection and post-collection data processing (but not analysis).

We will assume this setup for each arterial corridor that would be sampled in a future study. Assume that N arterial corridors of the same facility and area type are chosen for the purpose of estimating a speed-flow curve for that cell. Then, $10 \times N$ bi-directional links will be driven (5 links times 2 directions), and $320 \times N$ traverses will be performed in total. Based on the assumptions outlined above, the residual variance in the speed-flow curves will be:

$$\sigma^2 = 0.5 + \frac{\sigma_2^2}{10N} + \frac{\sigma_3^2}{320N} \quad (2)$$

Using the values for σ_2^2 and σ_3^2 derived from the existing floating car data, choosing $N = 3$ corridors will permit one speed flow curve to be estimated to a precision of 1.1 mph at an estimated cost of \$12,000. If $N = 4$ corridors are chosen, the speed-flow curve can be estimated to a precision of 0.9 mph at a cost of \$16,000. Exhibit 33 compares the tradeoff among sample size, precision and cost in this case.

Exhibit 33. Tradeoff of Sample Size versus Precision and Cost for Narrow A:B Comparisons

Corridors (N) per Speed- Flow Curve	Std Error (σ) of Estimate (mph)	Sampling Cost
3	1.1	\$12,000
4	0.9	\$16,000
5	0.8	\$20,000
6	0.8	\$24,000
7	0.7	\$28,000
8	0.7	\$32,000
9	0.6	\$36,000
10	0.6	\$40,000
11	0.6	\$44,000
12	0.5	\$48,000
13	0.5	\$52,000
14	0.5	\$56,000
15	0.5	\$60,000

Note: Sampling cost includes field data collection (traffic counts and vehicle speeds) and post-collection data processing costs only, and does not include costs related to the design of the study or speed-flow curve development. See Exhibit 32 on page 39 for a breakdown of unit cost assumptions.

For example, one could select 3 or 4 sites for each distinct speed-flow curve to be estimated if a precision of 1 mph were acceptable. If a total of 6 to 8 different speed flow curves were to be estimated, and a budget of \$100,000 were available for field work and post-collection data processing, one could select N = 3 or 4 corridors for each curve and sample a total of 24 or 25 arterial corridors. A study to update fewer curves could be done at a lower cost depending on the number of curves.

Optimizing the Data Collection Effort

When selecting corridors for the base update study, it would be possible to design sub-experiments to advance the understanding of a variety of issues regarding the relationship between speed and flow on the arterial system. In general, sub-experiments are designed by pairing a link to be sampled with another link in such a way as to form an A:B comparison. The subset of the data for the A:B pairs would be examined to answer the question(s) posed in the sub-experiment(s), while all of the data would still be used for updating the speed-flow curves.

Whether to design A:B comparisons in a future study, and which comparisons to make, would be determined at the time of the future

study based on the issues and questions that are then most important to SCAG. The comparisons are of two types, Narrow and Broad, with differing tradeoffs between sample size and precision. Each of these comparisons and its implications are discussed below.

Narrow A:B Comparisons

This type of comparison entails collecting data on a link previously driven and gives the most precise resolution of speed changes. Because data pairs are formed for the same links, only the variance associated with Component 3 (traverses within links) affects the determination of speed changes, and the first and second components of variance are eliminated.

In general, one would pair up hourly-average {average speed, V/C} data points collected at two points in time on the same link and, then, test for changes in speed after adjusting for differences in V/C. The result would be to determine whether a change has occurred in the speed-flow relationship over time. Examples of the use of narrow A:B comparisons include:

- Re-driving at least some of the links sampled in the 2004 study and conducting a paired analysis as described above to test for changes in the speed-flow relationship.
- Driving a selected corridor before and after a change in the configuration of the roadway system to test whether the change induced a shift in the speed-flow relationship. For example, this could be done to test the effect of an improved signal control technology.

Using assumptions consistent with those for Equation (2), if narrow A:B comparisons are formed for N corridors the residual variance of mean differences in speeds will be given by:

$$\sigma^2 = 2 \frac{\sigma_3^2}{320N} \quad (3)$$

where the factor of 2 is introduced by the presence of pairs of data points. Exhibit 34 demonstrates that this method can result in precise (but narrow) comparisons at small sample sizes. Note that the tabulated cost has already been counted in the total cost of data collection and processing for the base update study and does not present a new, additional cost. It should be used as a simple indicator of the emphasis being given to each sub-experiment that might be planned.

Exhibit 34. Tradeoff of Sample Size versus Precision and Cost for Narrow A:B Comparisons

Corridors (N)	Std Error (σ) of Estimate (mph)	Sampling Cost
1	0.61	\$ 4,000
2	0.43	\$ 8,000
3	0.35	\$12,000
4	0.30	\$16,000
5	0.27	\$20,000

Note: Sampling cost includes field data collection (traffic counts and vehicle speeds) and post-collection data processing costs only, and does not include costs related to the design of the study or speed-flow curve development. See Exhibit 32 on page 39 for a breakdown of unit cost assumptions.

Broad A:B Comparisons

The second type of comparison entails the pairing of different corridors belonging to the same facility and area type cell to test for differences associated with a characteristic that varies among the corridors. Because all of the 2004 driving was done in Los Angeles County, testing for the presence of county-specific differences in the speed-flow relationship is a primary recommendation for the next triennial model update cycle. This could be done by driving on corridors in other counties and forming broad A:B comparisons with the Los Angeles corridors driven in the 2004 study.

Broad A:B comparisons do not benefit from the variance reduction that results from comparing the same links. Thus, Equation (2) gives the variance of each of the data points in a pair, and the resulting variance of the comparison is given by 2 times Equation (2). Exhibit 35 demonstrates the extent to which this comparison requires larger sample sizes.

As noted for the narrow comparisons, the tabulated cost is only an indicator of the emphasis being given to each sub-experiment that might be planned and does not present a new, additional cost.

Exhibit 35. Tradeoff of Sample Size versus Precision and Cost For Broad A:B Comparisons

Corridors (N)	Std Error (σ) of Estimate (mph)	Sampling Cost
3	2.0	\$12,000
4	1.7	\$16,000
5	1.5	\$20,000
6	1.4	\$24,000
7	1.3	\$28,000
8	1.2	\$32,000
9	1.1	\$36,000
10	1.1	\$40,000
11	1.0	\$44,000

Note: Sampling cost includes field data collection (traffic counts and vehicle speeds) and post-collection data processing costs only, and does not include costs related to the design of the study or speed-flow curve development. See Exhibit 32 on page 39 for a breakdown of unit cost assumptions.

5.5 Proportionate Arterial Speed Sampling (Approach II)

The 2004 study was designed to collect arterial speed data for the specific purpose of updating arterial speed-flow curves in the SCAG transportation planning model. To this end, arterials were selected in a way that optimized the power of the data for estimating speed-flow curves, but that was not a proportionate sample of speed and V/C conditions on the SCAG system. Therefore, average speeds for the system cannot be estimated directly from the speed data, but must be estimated by combining the speed-flow curves with data on the incidence of V/C conditions on the system. This is done, for forecast purposes, by the SCAG model and could be done for an historical period as part of model calibration or by calculations outside the model.

In this section, we respond to a SCAG request to estimate the precision with which the system average speed could be estimated directly using floating car data. To do so, a study would select a random sample of arterial corridors throughout the SCAG system to drive across a range of time periods in order to sample the wide variety of V/C conditions that occur. Once a corridor was selected, it is likely that the data collection would be similar to the recently completed study, i.e., the ~5 links on the corridor would be driven in round trips averaging 15 minutes long and once deployed at a location, data would be collected for 4 consecutive hours.

The sampling would be allocated by time period in such a way as to correspond to the distribution of VMT by day of week and hour of day on the SCAG system. Two options could be used to obtain wide coverage of time periods:

- Return to the same corridor for 4-hour driving periods on different days of the week and hours of the day; or
- Randomize the choice of time period (day and hours) with the choice of corridor, so that more corridors are driven in total, but for no more than 4 hours each.

The first of these choices would support estimating the true average speeds on the individual corridors in addition to the systemwide average speed. Estimating speeds on individual corridors would not be possible with the second choice, but the systemwide average speed could be estimated with greater precision because more individual corridors would be covered. In the calculations that follow, we will assume that the sample design will be based on the second choice.

The proportionate sampling approach will require a larger sample size to achieve a given level of precision. In Approach I, other data are used to give the incidence of V/C conditions, so that the effect of varying V/C conditions need not be fully sampled in the floating car data. In Approach II, the floating car data must fully sample the range of V/C conditions; the variance encountered will be larger and more corridors must be driven to achieve a given level of precision.

For purposes of estimating the sample size versus precision tradeoff, we must judgmentally increase the variance observed in the existing data to account for the higher variance that would be encountered in proportionate sampling. All of the components of variance estimated in Section 2 for the existing data will apply, plus we must account for the wider variance in V/C conditions that would be encountered.

In the absence of data that would directly reflect proportionate sampling, we have developed bounding cases that assume the variance would be at least twice as large as in the existing data and possibly three times as large. If N corridors are driven covering the 10 facility and area type cells recognized on the SCAG system, and f is the assumed adjustment factor (2 or 3), then the variance of the system average speed will be given by:

$$\sigma^2 = f \left(\frac{\sigma_1^2}{10} + \frac{\sigma_2^2}{10N} + \frac{\sigma_3^2}{320N} \right) \quad (4)$$

Exhibit 36 displays the result of evaluating Equation (4) for the two cases in which the variance is assumed to be twice and three times that encountered in the existing data. The cost of data collection for a given value of N is the same in the two cases, but Case 2 will show lower precision than Case 1 for the estimated system average speed.

One can conclude from the table that if \$100,000 were expended on floating car driving (excluding study design or data analysis costs), the proportionate sampling approach would probably estimate the system average speed to a precision where the standard deviation (σ) is approximately 1 mph. The collected data could also be used to develop speed-flow curves, although the resulting curves would be subject to larger errors compared to that listed in the preceding section because the targeted method used in the recent study to select links increased the power of the data for determining speed-flow curves above that of proportionate sampling.

Exhibit 36. Tradeoff of Sample Size versus Precision and Cost For Proportionate Speed Sampling

Corridors (N)	Std Error (σ) of Estimate (mph)		Sampling Cost
	Case 1 $2 \times \text{Variance}$	Case 2 $3 \times \text{Variance}$	
8	1.6	2.0	\$32,000
9	1.5	1.9	\$36,000
10	1.4	1.8	\$40,000
11	1.4	1.7	\$44,000
12	1.3	1.6	\$48,000
13	1.3	1.6	\$52,000
14	1.2	1.5	\$56,000
15	1.2	1.4	\$60,000
16	1.1	1.4	\$64,000
17	1.1	1.4	\$68,000
18	1.1	1.3	\$72,000
19	1.1	1.3	\$76,000
20	1.0	1.3	\$80,000
21	1.0	1.2	\$84,000
22	1.0	1.2	\$88,000
23	1.0	1.2	\$92,000
24	0.9	1.1	\$96,000
25	0.9	1.1	\$100,000

Note: Sampling cost includes field data collection (traffic counts and vehicle speeds) and post-collection data processing costs only, and does not include costs related to the design of the study or speed data analysis. See Exhibit 32 on page 39 for a breakdown of unit cost assumptions.

5.6 Recommendations

This chapter has presented a framework to guide the design of future studies to collect arterial speed data for the SCAG system. From the information presented here, one can see that the \$100,000 cost of a study to estimate the system average speed by sampling 25 corridors using Proportionate Sampling (Approach II) would be similar to the

cost for updating speed-flow curves under Approach I in the event that 6-8 different speed-curves were to be estimated. Which approach is to be preferred depends on factors that are not known at present and on the importance placed by SCAG on obtaining direct measurements of system speed versus the minimum task of updating speed-flow curves on a triennial basis in conjunction with RTP development.

An important unknown factor is the number of distinct speed-flow curves that are needed to adequately characterize the SCAG arterial system. If the curve is defined so that actual speed is normalized by free flow speeds that vary by facility and area type, then it is at least theoretically possible that only one speed-flow curve could be needed. In general, one can observe that:

- Approach I offers the potential for updating the speed flow curves at what could be a substantially lower cost if the number of distinct speed-flow curves is small compared to the number of facility and area type cells.
- Approach I and II would be of comparable cost if the number of distinct speed-flow curves is as many as 6 to 8. In this event, Approach II offers the advantage of providing for both the updating of speed-flow curves and the development of an independent estimate of system average speed to a precision of approximately 1 mph.

SCAG's decision on which approach to use must consider the outcome of its analysis of the existing data, the budget for future update studies, and the importance to be placed on the independent estimate of the system-wide average speed.

In the event that Approach I is taken for the next study, we strongly recommend that a substantial portion of the driving be allocated to counties outside Los Angeles, in order to test for the presence of county-specific differences in the speed-flow relationship. It might be appropriate to allocate a small portion of the driving to corridors already covered in the 2004 sampling, but we would recommend allocating substantially more than half of the sample to corridors in other counties. This allocation would allow for detection of county effects and also broaden coverage of the SCAG system.

An example of this approach is as follows. Assume that 25 links were to be driven. Because a narrow A:B comparison involving a single link can detect a speed change as small as 0.6 mph, one might choose to re-drive one corridor sampled in the 2004 data collection in each of 3 to 5 facility and area type cells that account for the largest share of VMT. From this, one could obtain some understanding of the stability of the speed-flow relationship over time. The remaining 20 to 22 corridors could be allocated to corridors in Los Angeles and the other counties (not sampled in 2004) so the data would be distributed by county in a manner that approximates the distribution of VMT.

If Approach II is chosen for the next study, a sample of about 25 corridors and a data collection budget of \$100,000 will be required to achieve the 1 mph level of precision. Because sampling in this approach is proportional in nature, it will naturally give proportionate coverage of counties, time periods, V/C conditions, and other factors in the data.

6 APPENDIX – SPEED DATA COLLECTION TECHNOLOGIES

There are a wide variety of speed data collection methodologies and technologies that can be difficult to differentiate. However, if we look on them as “sampling” techniques for measuring speeds from the universe of vehicle speeds present on an arterial, it becomes possible to group them into categories and assess their inherent strengths and weaknesses.

The universe of arterial speeds covers three dimensions, time, space, and the vehicles (see Exhibit 37). There are no arterial speed data collection techniques that currently have the capability to sample 100% in all three dimensions. Each technique usually samples 100% in only one or two of the dimensions.

For example, roadside detectors, such as loop or video detectors measure the speeds of most all vehicles passing by, all day, (2-dimensions) but they provide this information for only the points where the sensors are located. There is no information on the speeds in between sensors.

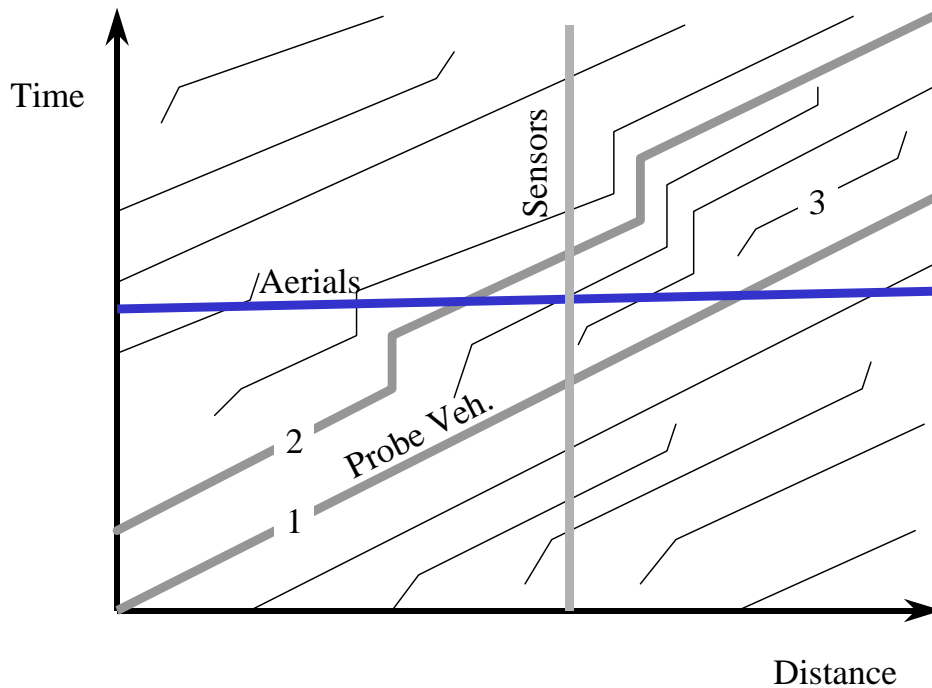
Aerial photography can measure the speeds of all vehicles everywhere on the arterial (2-dimensions), but only for a few minutes at a time, thus limiting the amount of time for which the data can be collected.

Probe vehicles (the thick gray diagonal lines in the figure) can measure speeds over both distance and time (2 dimensions), but they are limited to a small sample of vehicles. By understanding how each technique samples the arterial speed universe we can understand their basic strengths and weaknesses.

There are a multitude of technologies for measuring arterial speeds but they fall into three basic categories, according to their particular method of sampling the arterial speed universe.

- 1) Fixed-Location (Roadside) Detectors, which measure the speeds of all vehicles at specific locations for long periods of time (the sensors in Exhibit 37);
- 2) Aerial Photography, which measures the speeds of all vehicles at a specific time over the full length of the arterial; and
- 3) Vehicle Tracing Techniques (Probe Vehicles in Exhibit 37), which follow selected vehicles over the length of the arterial.

Exhibit 37. Vehicle Trajectories on an Urban Street



Fixed-Location (Roadside) Detectors

Roadside detectors are permanent or portable instruments located within or alongside the roadway for measuring the spot speeds of vehicles passing selected points of the roadway. Technology variations of this technique include: single loop detection, double loop detection, loop detectors with vehicle signature matching, portable tubes, radar/microwave/infrared, laser and video detectors.

Single Loops: Single loops count vehicles and measure occupancy (the amount of time a vehicle spends on the loop). A typical single loop is 6 feet wide and 6 feet long, with one loop located in each travel lane. The average speed is calculated from count and occupancy data assuming a value of the effective vehicle length (the average length of the vehicle plus the detector length or g factor). The g factor varies by time of day and travel lane, and a constant value produces inaccurate speed estimates. The PeMS system which collects and stores loop detector data from all California freeways uses an adaptive algorithm to compute the g-factor.

Loops are typically located at 0.3 to 0.5 mile intervals. On arterial links loops are placed mid-block approximately 300-400 ft upstream of the intersection stopline. The speed is measured at the detector locations and extrapolated across the rest of the link. The extrapolation from a point to a line is not always representative of the mean travel speed over the length of a facility. For example, at the

beginning of congestion, the speed variations along the length of a link can be quite large.

Double Loop Detectors: Double loop detectors provide much more precise measurements of vehicle speeds than single loop detectors. Double loops consist of two 6 foot wide loops located 14 feet apart, one pair of loops in each travel lane. With a double loop, the vehicle speeds are directly measured by dividing the distance between each loop in the pair by the time it takes the front edge of a vehicle to travel between each loop in the pair.

Vehicle and Platoon Signature Matching (“advanced loops”): Average speed (travel time) is calculated by matching or correlating unique vehicle signatures between sequential observation points. Vehicles with unique signatures among passenger vehicles include large trucks, buses, and various truck/trailer combinations. Platoon matching uses similar concepts, except that this method correlates the number of large vehicles within a vehicle platoon between two consecutive locations.

The approach uses conventional loop detector systems equipped with advanced detector cards to process the detector signals to obtain vehicle signatures. Recent field tests at a signalized intersection in Irvine, CA showed that this method produced travel times within 10 % of true travel times for through and right turn vehicles.

Ongoing research explores platoon matching between upstream and downstream detectors on an arterial link, using conventional loop detectors and second-by-second processing of the loop data. Travel time is calculated as the difference in the arrival times of the median vehicles in the platoon.

Radar/Microwave/Infrared Detectors: Radar, microwave, and infrared detectors are mounted along the side of the road or overhead. An approaching vehicle is irradiated with radar, microwaves, or an infrared beam. The Doppler shift in the frequency of the reflected beam is used to estimate the vehicle speed. Some of these detectors cannot detect stopped vehicles (i.e., vehicle presence) which is essential for monitoring traffic conditions. The remote traffic microwave sensor (RTMS) has been employed in several locations to replace loop installations. A single side-mounted RTMS unit can provide data for up to four lanes of traffic.

Laser Detector: A prototype detector system for measuring travel times has been recently developed . The system is mounted above a roadway. Two laser units project two laser beams to the roadway. The reflected light from each laser beam is detected by an associated sensor. Each sensor converts the laser signal into an electronic signal, which is then processed by electric circuits and a microprocessor. The signals are processed to measure vehicle lengths and speeds.

Video Detectors: Video cameras and vision image processing algorithms have been used as detection devices. Video detectors are either loop emulation systems or track vehicles. The Autoscope system is a loop emulation system. Users place lines on the field of view and the system measures flows, occupancies and speeds at the user designated locations. The system has been used to provide detection on several signal control systems (e.g., vehicle actuation). Vehicle tracking systems track vehicles across the pixel elements within the field of view and produce vehicle trajectories.

Key considerations in deploying video detection is the proper location of cameras (height and angle) to avoid occlusion problems (traffic in one lane from blocking the visibility of other lanes), and potential problems of reflected and direct sunlight, fog and rain.

Aerial Photography

Aerial photography has been used for several years to measure congestion, but is a relatively recent innovation for measuring vehicle speeds over large areas. Large amounts of data over large geographic areas can be gathered very quickly. Technology variations of aerial photography include: time lapse photography, vehicle density photography, digital cameras, digital scanning, and satellites.

The prime disadvantage is the labor involved in identifying and counting the vehicles in the photos. Another disadvantage is that aerial photography is feasible only when minimum requirements for lighting and visibility are met. For example, low level fog or clouds (below 9,000 feet), and high winds can prevent the aircraft from either being able to get a good image of the ground or to stay on the desired course. Also, aerial photography requires sunlight. Long shadows also interfere with vehicle detection in the photos.

Time lapse photography involves taking two or three photographs a few seconds apart. With knowledge of spatial scales and of the exact time between photographs, the change in position of a vehicle in two photographs gives its speed. In addition, vehicle type and lane can be identified. Photos are typically shot at about 12000 to 1 scale (one inch equals one thousand feet).

Vehicle Density Photography: Vehicle density photography requires fewer photos than time lapse photography, and it is not so critical to be able to reliably scale ground distances in the photos. The density of vehicles is determined simply by counting the number of vehicles present on a section of the freeway and dividing by the number of lanes and the length of the section. The speed is calculated from a speed-density equation specifically calibrated for the area and facility type.

The results from a field study show that the aerial photo estimated mean speeds were within 1.4 mph of the floating car estimated speed

when time lapse photography is used. Density photography (which depends upon a calibrated speed-density equation) is less accurate, yielding mean speeds with a standard error of plus or minus 5 mph. The much larger sample sizes possible with aerial photography however allow a better estimate of the variation in speeds on the facility than can be obtained from floating cars.

The manual identification of vehicles in each photo is quite labor intensive. Digital cameras offer the opportunity to generate computer readable images directly that might be processed with image processing software. Images are captured and stored as computer-readable files. Digital cameras, however, cannot yet achieve the same resolution and range of light sensitivity as film.

The technology of direct digital photography may have great promise, but it does not appear to be mature enough yet for use in aerial photography.

As an alternative to the use of direct digital photography, digital scanning of photos was tested in this project to determine the feasibility of reducing the labor costs involved in processing time lapse photos.

Digital scanning was quite successful. Photographic image quality was perceptibly better two hours before sunset, however the shadow of trees and buildings at that relatively low sun angle made vehicle identification difficult in some circumstances. The images recorded after sunset required the use of a slow (1/125 s) shutter speed, and were still underexposed by approximately two stops (i.e., a factor of four). In spite of the potential loss of sharpness and the underexposure, these images were perhaps more usable than the earlier ones due to the lack of shadows. This finding also suggests that imagery collected during overcast conditions may in fact be more useable than that collected under clear skies.

Vehicle Tracing Techniques

Vehicle tracing techniques involve tracking either test vehicles or randomly selected vehicles through a network of checkpoints to determine the travel times between points. The vehicles being traced may be private automobiles, transit vehicles or trucks.

Test Vehicle (Floating Car) Technique

The test vehicle method (often referred to as floating car) has been the most common travel time collection method employed to date. This method consists of driving a vehicle along a pre-selected route and measuring the elapsed time and distance traversed.

Data collection personnel within the test vehicle control the speed of the vehicle according to set driving guidelines (i.e., average car, floating car, and maximum car), although it is usually the median travel time that is measured. In this latter case, the “floating car” is driven so that it is passed by as many vehicles as it passes during the run.

Travel time, speed, and delay information can be recorded in the test vehicle manually or using Electronic Distance Measuring Instrument (DMI), and Global Positioning System (GPS) Receiver.

The manual method requires two people per car. The passenger in the test vehicle manually records travel times at designated checkpoints using a clipboard and stopwatch. The manual method does not require any significant initial capital investment in equipment, but it is difficult to control the quality of the data and the manual method does not provide information on how the speed profile varies between checkpoints.

The DMI method requires that specialized equipment be purchased and installed in the test vehicle. An electronic DMI is connected by a special cable to the vehicle’s transmission. The DMI is then coupled with a portable computer and specialized software to record speeds and distances traveled up to every half-second or greater. The DMI must be recalibrated every time the tires are changed or the tire pressures are adjusted on the test vehicle. Only one operator is required, so labor costs are typically half those of the manual method. The DMI method eliminates data recording errors and provides speed profiles between checkpoints.

The GPS method requires that specialized equipment be purchased and a receiving antenna be installed in the test vehicle. A GPS receiver coupled with a portable computer is used to record the test vehicle’s position and speed at time intervals as frequent as every second. GPS equipped floating cars and software are increasingly being used in travel time studies with reported position accuracy of 2 ft.

Non-Instrumented Vehicle Tracking Technique

This technique uses any one of several technologies for identifying randomly selected un-instrumented vehicles in the traffic stream at various checkpoints within the study area and measuring the time between appearances at each checkpoint.

License Plate Matching: The license plate matching method consists of collecting vehicle license plate characters and arrival times at sequential checkpoints, matching the license plates between checkpoints, and computing travel times from the difference between arrival times at checkpoints. License plate matching has been used by

relatively few transportation agencies for travel time data collection, even though license plates are commonly collected for origin-destination and through traffic surveys. Potential reasons for this low use may include the difficulty of reading full license plates at highway speeds, recording arrival times using manual methods, or the inability to control those vehicles providing the travel time data. Character recognition software may be used to reduce the labor costs associated with the manual transcription of license plate characters from the video recording for subsequent computer matching.

Advanced Vehicle Identification (AVI) Systems: The Advanced Vehicle Identification (AVI) system uses a combination of electronic tags on board of a private or publicly owned vehicle, and roadside readers to track vehicles along a given roadway or transportation link. A centralized system then matches the tagged vehicles and calculates a series of roadway conditions, including vehicle speeds and travel time.

The TravInfo project in the Bay Area uses readers to track vehicles equipped with electronic toll collection (ETC) and calculate travel times between the reader locations. Similar technology is used in Houston and New York.

Cellular Phone Tracking/Geolocation Systems: Geolocation techniques can provide a relatively inexpensive and accurate method for estimating traffic conditions over a region. The geolocation technique automatically detects phone call initiation, and will locate a given vehicle within a few seconds. After it has determined that a vehicle is on a roadway of interest, the system will periodically plot the vehicle's location to determine its speed and travel distance. It will average the speeds of numerous vehicle on a give roadway and can automatically estimate the travel time along a given corridor, during a given time period. This approach received increased attention following the Federal Communications Commission (FCC) mandate of E911 requirements (i.e., cellular phone location be provided within 150-900 ft for emergency calls). However, cellular phone based probe vehicles are not yet deployed despite promising results through simulation and field tests.